City of Bryan

Drainage Design Guidelines

2003

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Section 1

Introduction

1.1 REVIEW PROCESS

The review process for any drainage plan involves a two- or three-step process. The preliminary drainage conference with the City engineering staff is encouraged, and a final Drainage Report or Abbreviated Drainage Plan is required for all Grading Permits. If desired, a Preliminary Drainage Dlan may be submitted. In addition, a Grading Permit application must be completed and submitted to the City for approval prior to grading the site.

1.1.1 TxDOT PERMIT

If the development is located near a TxDOT right-of-way and storm water runoff from the development enters the TxDOT right-of-way, then five (5) copies of a TxDOT utility permit application will need to be submitted to the City of Bryan. After the City of Bryan reviews the permit application, the application will be forwarded to TxDOT for approval. The application needs to be accompanied by all relative drainage calculations sealed by an engineer licensed in the State of Texas.

1.2 GENERAL DRAINAGE PLAN REQUIREMENTS

The submitted drainage plan must satisfy the requirements of the City of Bryan Stormwater Management Ordinance No. 669 and the Design Guidelines contained herein.

Additionally, in accordance with State Law TX WATER § 11.086, V.T.C.A., Water Code § 11.086, no development shall be designed, constructed or maintained so as to unreasonably:

a. Impede the natural flow of surface water from higher adjacent properties;
b. Alter the natural flow of surface waters so as to discharge them upon adjacent properties at a more rapid rate, in greater quantities or in a different location than would result from the predevelopment natural flow of surface waters; or
a. Collect or concentrate the flow of surface waters for discharge into an existing natural or artificial drainage way in a manner, which exceeds the capacity of the receiving watercourse.

1.3 PRELIMINARY DEVELOPMENT CONFERENCE

The drainage design concept for the proposed development may be discussed with the City engineering staff prior to the development of any specific design or plan preparation. This optional conference is offered to help guide the drainage design and minimize the review time. The conference may address the information relative to the proposed development shown in the list below.

Items in bold are more essential to have for the conference.

1. General Location
    a. Local streets, within and adjacent to the development
    b. Major drainage ways and drainage structures in the vicinity
    c. Names of surrounding developments
2. Property Description
    a. Acreage
    b. Type of ground cover (i.e., forest, pasture, etc.)
    c. Name of owner and type of development
3. Major Basin Description
    a. Major basin drainage characteristics
    b. Related previous drainage studies
    c. Flood insurance rate maps
4. Sub-Basin Description
    a. Discussion of offsite drainage flow patterns and anticipated impact on development
    b. Discussion of historic drainage patterns of the area proposed for development
5. Drainage Facility Design
    a. General Concept
        (1) Discussion of concept and typical drainage patterns
        (2) Discussion of compliance with offsite runoff considerations
        (3) Discussion of the content of tables, charts, figures, or drawings presented in any reports
    b. Specific Details
        (1) Discussions of drainage problems encountered and solutions at specific design points
        (2) Discussion of detention storage and outlet design
        (3) Discussion of maintenance access and aspects of the design
6. References
    a. Reference all criteria master plans, and technical information used in support of concept.
7. Drawings
    a. Sheet #1 - General Location Map that:
        (1) Depicts drainage flows entering and leaving the development site
        (2) Identifies major construction along path of drainage
        (3) Illustrates general drainage flow within entire basin and
        (4) Is drawn at a scale of up to 1"=2,000'
    b. Sheet #2 - Floodplain Information
        (1) Copies of currently effective City of Bryan flood insurance rate maps showing the location of the subject development.

      c.   **Sheet #3 - Drainage Plan on a 24" x 36" sheet drawn at a scale of 1"=20' to 1" = 200' that illustrates the following:**
          (1)   **Existing topographic contours at 5-foot maximum intervals**
          (2)   **Property lines and easements with purposes noted.**
          (3)   **All existing drainage facilities.**
          (4)   **Approximate flooding limits based on available information.**
          (5)   **Proposed drainage facilities including any retention or detention provided.**
          (6)   **Major drainage watershed boundaries and sub-boundaries.**
          (7)   **Any offsite features influencing development.**

1.4   DRAINAGE REPORT

Two copies of the Drainage Report shall be prepared and submitted along with all drainage plans for initial review. Upon approval the City will require two copies to keep and file. Additional copies, which may be needed and returned to the Engineer, will need to be submitted, approved, and returned. The purpose of the Drainage Report is to identify and define conceptual solutions to problems (onsite or offsite) as a result of the development. All reports shall be typed on 8 1/2" x 11" paper and bound together. The report shall include a cover letter presenting the preliminary design for review and shall be sealed by an engineer licensed in the State of Texas. The report shall contain the following certifications:

> "I hereby certify that I am familiar with the adopted ordinances, regulations, standards, and policies of the City of Bryan governing development, that these plans have been prepared under my supervision, and that this drainage plan complies with all governing ordinances and regulations to the best of my knowledge."
>
> "**(No Part/ Part)** of this site lies within the established area of special flood hazard as established by the current flood insurance study or flood hazard boundary map number _____."
> Dated:_____

       _____
       Licensed Professional Engineer
       State of Texas No. _____
       (Affix Seal)

1.4.1   DRAINAGE REPORT CONTENTS

The Drainage Report shall be in accordance with the following outline and contain the information listed:

1.   General Location and Description
   a.   Location
      (1) Local streets within and adjacent to the development
      (2) Major drainage ways and facilities
      (3) Names of surrounding developments
   b.   Description of Property
      (1) Acreage
      (2) Ground cover (forest, pasture, etc.)
      (3) Major drainage ways
      (4) General project description
2.   Drainage Basins and Sub-Basins
   a.   Major Basin Description
      (1) Reference to major drainage way planning studies such as flood hazard delineation reports and flood insurance rate maps
      (2) Major basin drainage characteristics.
   b.   Sub-Basin Description
      (1) Discussion of offsite drainage flow patterns and impact of development
3.   Drainage Design Criteria
   a.   Regulations: Discussion of optional criteria selected or deviation from this Drainage Design Guideline Manual, if any
   b.   Discussion of previous drainage studies for the site in question that influence or are influenced by the drainage design and how they will affect drainage design for the site
   c.   Discussion of the drainage impact of site constraints such as streets, existing structures, and development or Site Plan
   d.   Hydrologic Criteria
      (1) Identify design rainfall
      (2) Identify runoff calculation method
      (3) Identify detention discharge and storage calculation method
      (4) Discussion and justification of other criteria or calculation methods used that are not presented or referenced herein
   e.   Hydraulic Criteria
      (1) Identify various capacity references
      (2) Identify detention outlet type
      (3) Discussion of other drainage facility design criteria used that are not presented herein

4. Drainage System Design
   a. General Concept
      (1) Concept and typical drainage patterns
      (2) Compliance with offsite runoff and illegal diversion considerations
      (3) The content of tables, charts, figures, or drawings presented in the report
      (4) Anticipated and proposed drainage patterns
   b. Specific Details
      (1) Drainage problems encountered and solutions at specific design points
      (2) Detention storage and outlet design
      (3) Maintenance access and aspects of the design
5. Conclusions
   a. Comparison with Standards herein
   b. Effectiveness of drainage design to control damage from storm runoff
6. References - Reference all criteria and technical information used
7. Appendices (where applicable)
   a. Hydrologic Computations
      (1) Land use assumptions regarding adjacent properties
      (2) Minor and major storm runoff at specific design points for indicated design frequencies
      (3) Historic and fully developed runoff computations at specific design points
      (4) Hydrographs at critical design points
   b. Hydraulic Computations
      (1) Culvert capacities
      (2) Storm sewer capacity
      (3) Street capacity
      (4) Storm inlet capacity including inlet control rating at connection to storm sewer
      (5) Open channel design
      (6) Detention area/volume capacity and outlet capacity calculations
8. Drawings
   a. Sheet #1 - General Location Map that (size 8 ½ x 11):
      (1) Depicts drainage flows entering and leaving the development site
      (2) Identifies major construction along path of drainage
      (3) Illustrates general drainage flow within entire basin
      (4) Drawn at a scale of 1"=500' to 1"=10,000'
   b. Sheet #2 - Floodplain Information (size 8 ½ x 11):
      (1) Copies of currently effective City of Bryan Flood Insurance Rate Maps showing the property bounds of the subject development

    c.   Sheet #3 - Drainage Plan: Map(s) of the proposed development at a scale of 1"=20' to 1"=200' on a 24" x 36" (or 11" x 17" if legible) drawing shall be included. The plan shall show the following:

    (1) Existing and proposed contours at 2-foot maximum intervals

    (2) Property lines and easements with purposes noted

    (3) Streets

    (4) Existing drainage facilities and structures, roadside ditches, drainage ways, gutter flow directions, and culverts. All pertinent information such as material, size, shape, slope, and location shall also be included

    (5) Overall drainage area boundary and drainage sub-area boundaries

    (6) Proposed type of street flow (i.e., vertical or combination curb and gutter), roadside ditch, and gutter flow directions

    (7) Plans and profiles of proposed storm sewers and open drainage ways, including inlets, manholes, culverts, and other appurtenances

    (8) Proposed outfall point for runoff from the developed area and facilities to convey flows to the final outfall point without damage to downstream properties

    (9) Routing and accumulation of flows at various critical points for the minor storm runoff

    (10) Path(s) chosen for computation of time-of-concentration

    (11) Details of detention storage facilities and outlet works

    (12) Location and elevations of all defined floodplain affecting the property

    (13) Location and elevations of all existing and proposed utilities affected by or affecting the drainage design

    (14) Routing of offsite drainage flow through the development

1.5 CONSTRUCTION DRAWINGS AND SPECIFICATIONS

Where public drainage improvements are to be constructed in accordance with the approved Drainage Report, the Construction Plans (on 24" x 36" sheets) and specifications shall be submitted for review and approval prior to construction. Plan and profile sheets are required for the majority of improvements.

Information required for the drawings and specifications shall be in accordance with sound engineering principles, this Drainage Design Guideline Manual and City requirements. Construction documents shall include geometric, dimensional, structural, foundation, bedding, hydraulic, landscaping, and other details as needed to construct the storm drainage facility. The approved Final Drainage Plan shall be included as part of the construction documents for all facilities affected by the drainage plan.

The plans and specifications for the drainage improvements will include:

1. Storm sewers, inlets, and outlets
2. Culverts, end sections, and inlet/outlet protection
3. Channels, ditches, and swales
4. Erosion control facilities
5. Detention or retention pond grading, pilot channels, and outlets
6. Other drainage related structures and facilities
7. Maintenance access facilities and provisions
8. Finished floor elevations of adjacent buildings
9. 100-year water surface elevations
10. Engineer certification (*Section 1.4*)
11. Approval by the City

1.6   AS-BUILT DRAWINGS

A professional engineer licensed in Texas shall attest all public improvements and all detention facilities (private or public) constructed in accordance with an approved drainage plan by submitting plans reflecting as actual construction (As-Builts). The As-Builts shall be submitted on 24" x 36" three-mil film mylar to the City of Bryan before such improvements will be accepted for maintenance or prior to the issuance of a Certificate of Occupancy for any structure constructed on the site described by the drainage plan. Each sheet of the As-Built drawings must contain the certifications described in Section 1.6.1.

1.6.1   CERTIFICATIONS

1.6.1.1 Detention Facility Certification

As specified in Section 10-102 of the Flood Prevention and Protection Ordinance Chapter 10, the drainage plan, but not the abbreviated drainage plan, shall be constructed under the general supervision of a licensed professional engineer. The engineer shall submit an "as-built" plan to the City bearing the signature and seal of the engineer and the following certification statement:

> "I hereby certify that I am familiar with the approved drainage plan and associated construction drawings and furthermore, certify that the drainage facilities have been constructed in accordance with the approved construction plans or amendments thereto approved by the city to the best of my knowledge."

> _____
> Licensed Professional Engineer
> State of Texas No. _____
> (Affix Seal)

1.6.1.2 Public Infrastructure Certification

As specified in Section 25-8.D.3 of the Subdivision Regulations Ordinance Chapter 25, the As-Built drawings shall be prepared by the design engineer, under the guidance of the contractor, and shall bear a certification from the design engineer as follows:

> "(date) To the City of Bryan: I certify that the subdivision improvements shown on this sheet reflect any revisions of desgn of which I authorized, and/or any and all field changes of which I am aware."

> _____
> Licensed Professional Engineer
> State of Texas No. _____
> (Affix Seal)

Each sheet shall also bear a certification from the General Contractor as follows:

> "(date) To the City of Bryan: I certify that the subdivision improvements shown on this sheet were actually built, and that said improvements are substantially as shown hereon. I further certify, to the best of my knowledge, that the materials of construction and sizes of manufactured items, if any are stated correctly hereon."

> _____
> General Contractor

1.7 ABBREVIATED DRAINAGE PLAN – UNDER 1 ACRE COMMERCIAL LOT DEVELOPMENT

A site plan not requiring a detention pond or a building permit application for an existing lot shall include an Abbreviated Drainage Plan. The plan shall be submitted on a sheet at least 8 1/2" x 14", be drawn at a scale not less than 1"=20 feet, and include the following information:

1. Curb elevation adjacent to the lot, if applicable,
2. Elevations of front property corners if curb is not present,
3. Elevations of rear property corners,
4. Lowest proposed finished floor elevation,
5. Location of existing and proposed drainage swales and any easement associated with those swales,
6. Arrows depicting direction of flow of storm water runoff on and adjacent to the lot,
7. Boundary of the Area of Special Flood Hazard, if applicable, and
8. Source and date of effective Flood Insurance Study or other information regarding 100-year floodplain.
9. Contours at 2 foot intervals (existing and proposed)

1.8 WATERSHED DESIGNATION AND DESIGN STANDARDS

Nine (9) watersheds are found within the City of Bryan and its extraterritorial jurisdiction:

1. Overall Drainage Basin Map (Figure 1.1)
2. Briar Creek (Figure 1.2)
3. Burton Creek (Figure 1.3)
4. Carters Creek (Figure 1.4)
5. Cottonwood Branch (Figure 1.5)
6. Hudson Creek (Figure 1.6)
7. Still Creek (Figure 1.7)
8. Thompson's Branch (Figure 1.8)
9. Thompson's Creek (Figure 1.9)
10. Turkey Creek (Figure 1.10)

The delineation for the watersheds listed are approximate as well as some of the other data on the maps such as City Limits. Design storm criteria for each watershed are shown in Table 1.1. The City Engineer shall resolve any questions or complications concerning the location of a development within a watershed.
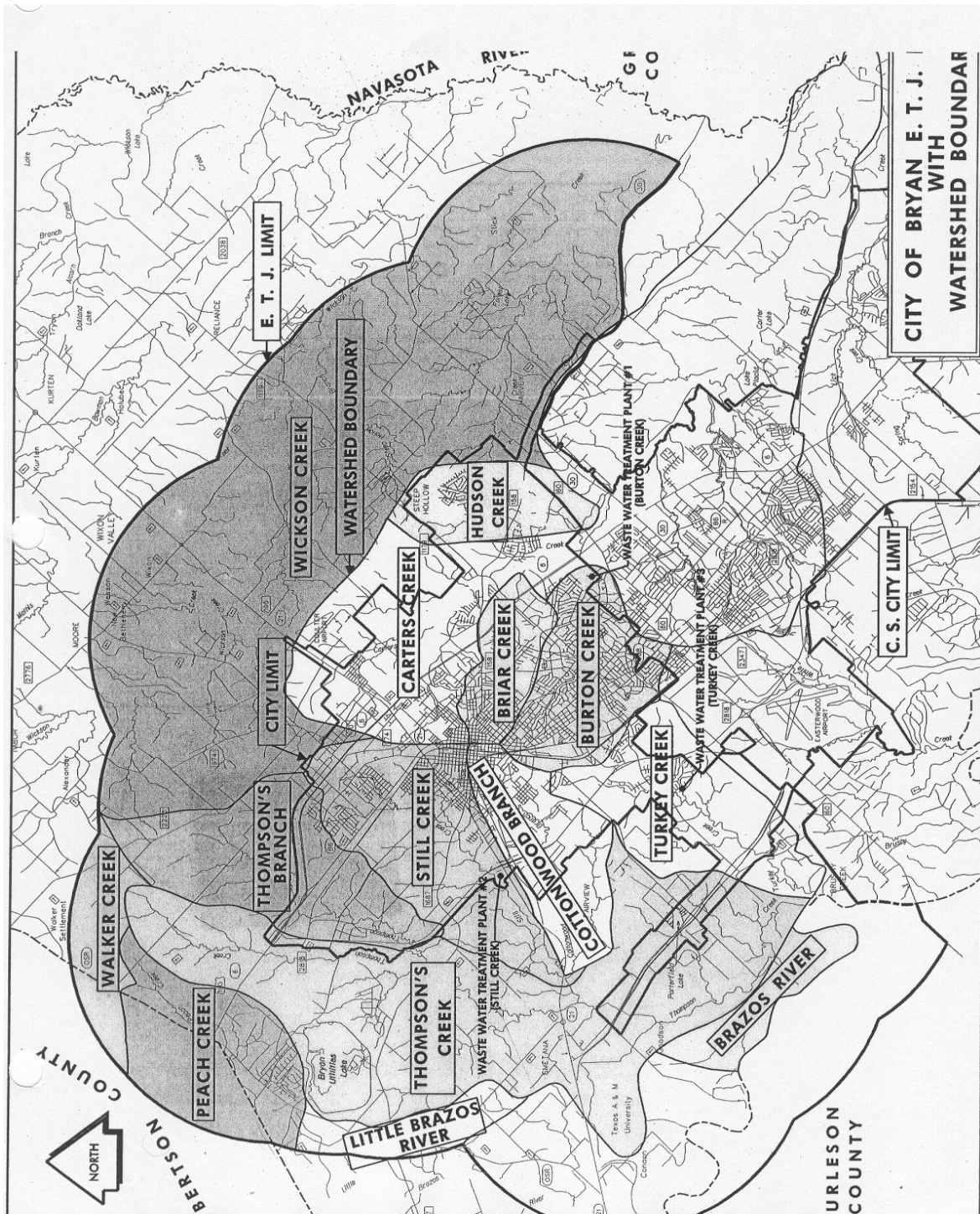
Figure 1.1 City of Bryan Drainage Basins
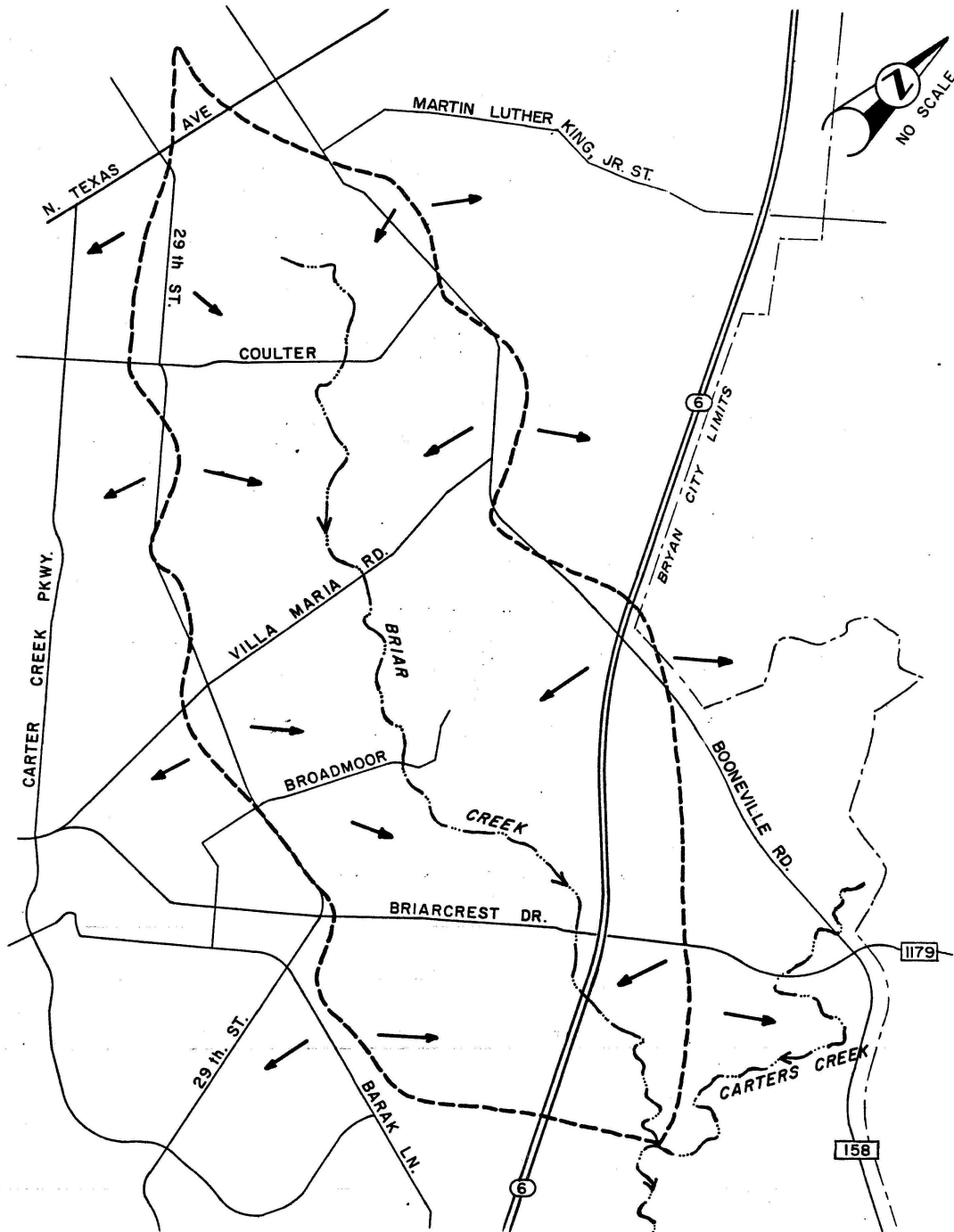
Figure 1.2
BRIAR CREEK DRAINAGE BASIN
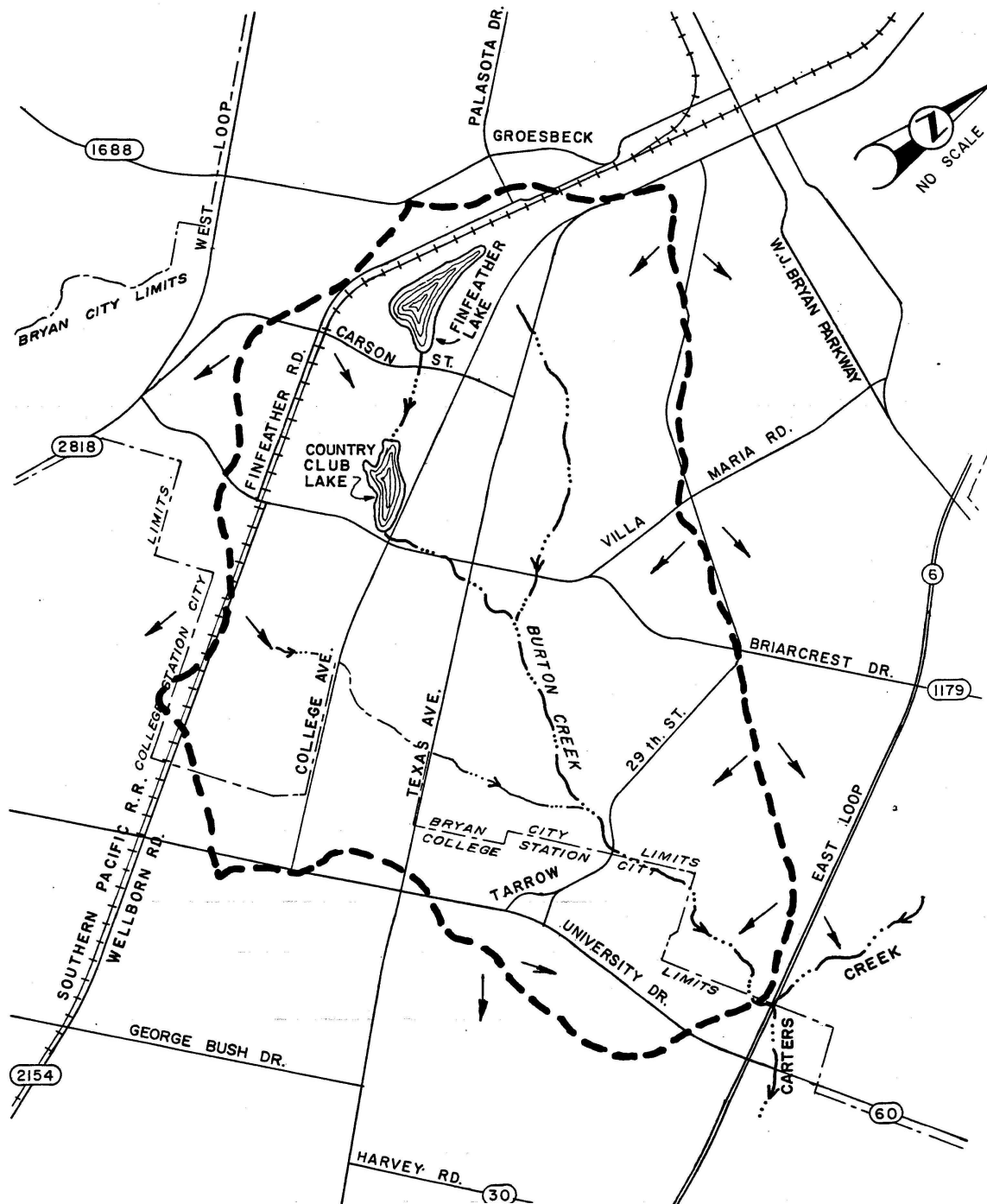
Figure 1.3

BURTON CREEK DRAINAGE BASIN
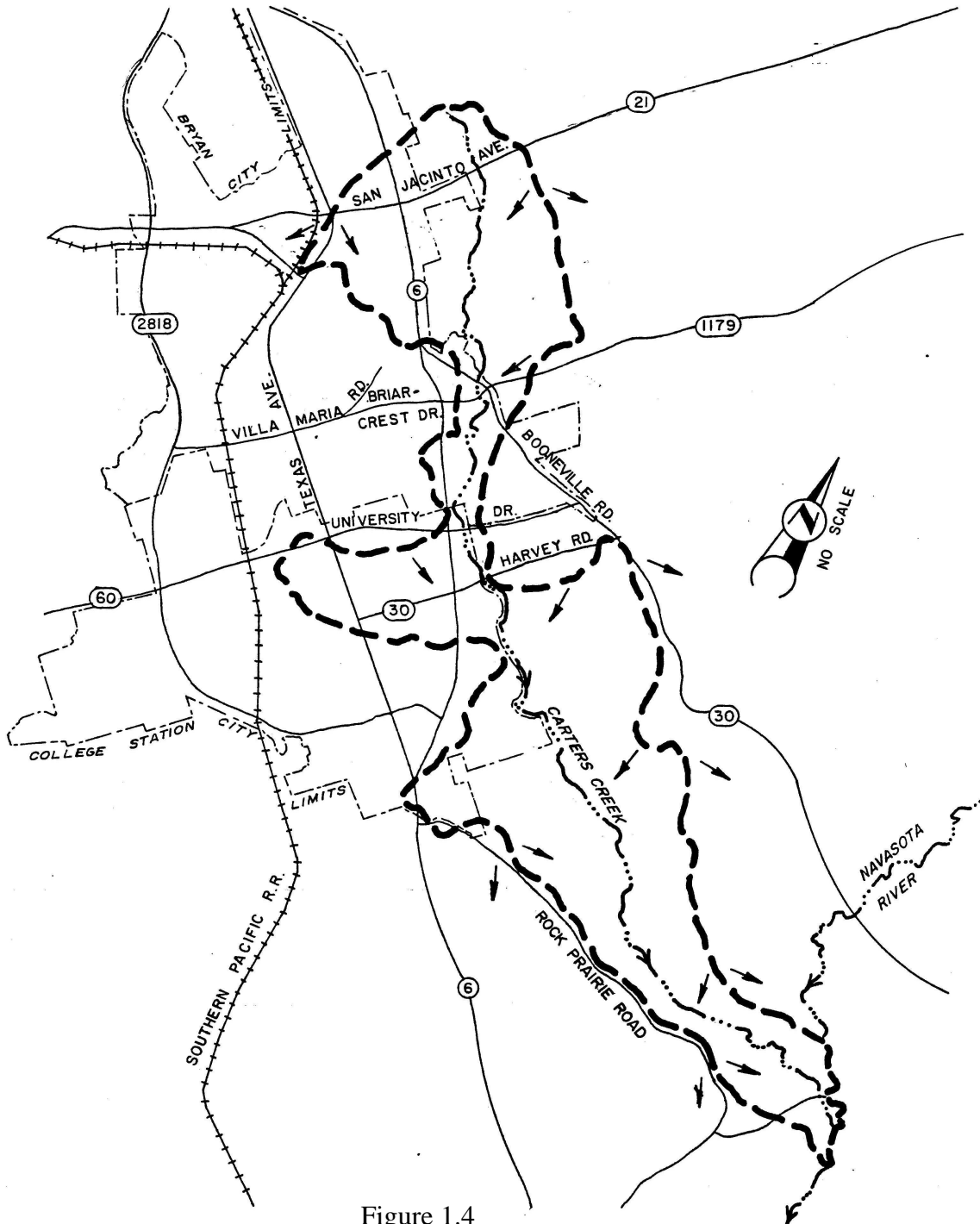
Figure 1.4

CARTERS CREEK DRAINAGE BASIN
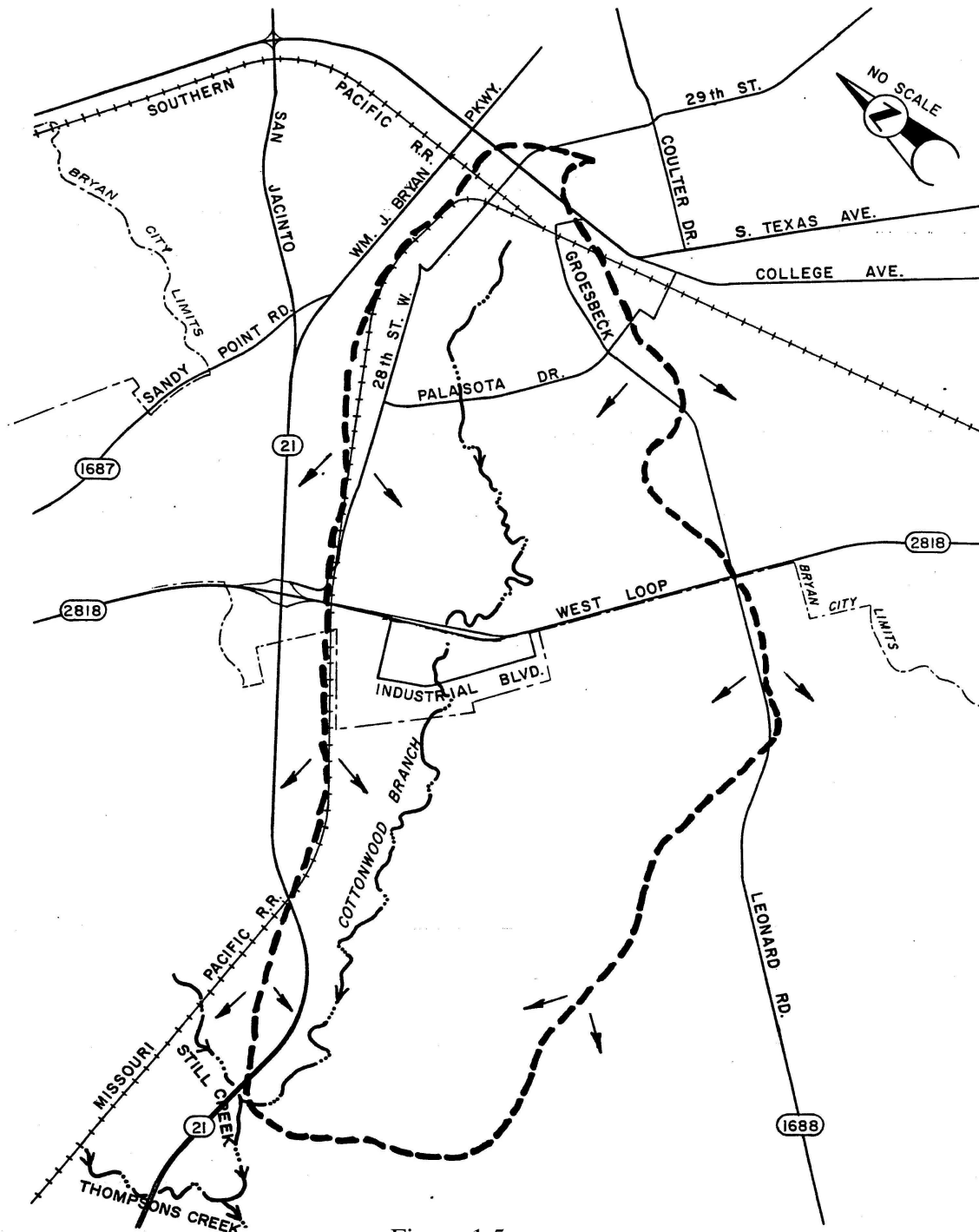
Figure 1.5

COTTONWOOD BRANCH DRAINAGE BASIN
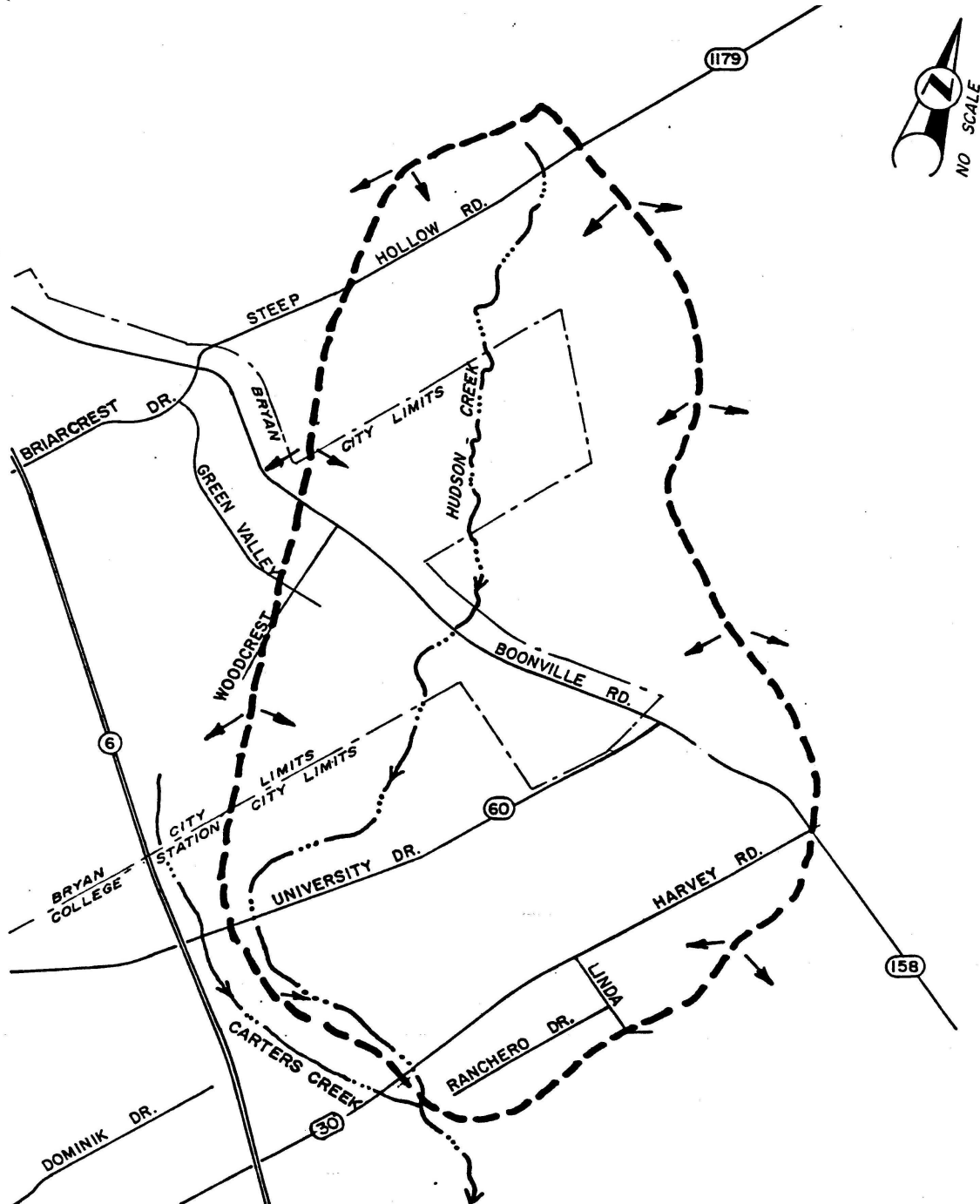
Figure 1.6

HUDSON CREEK DRAINAGE BASIN
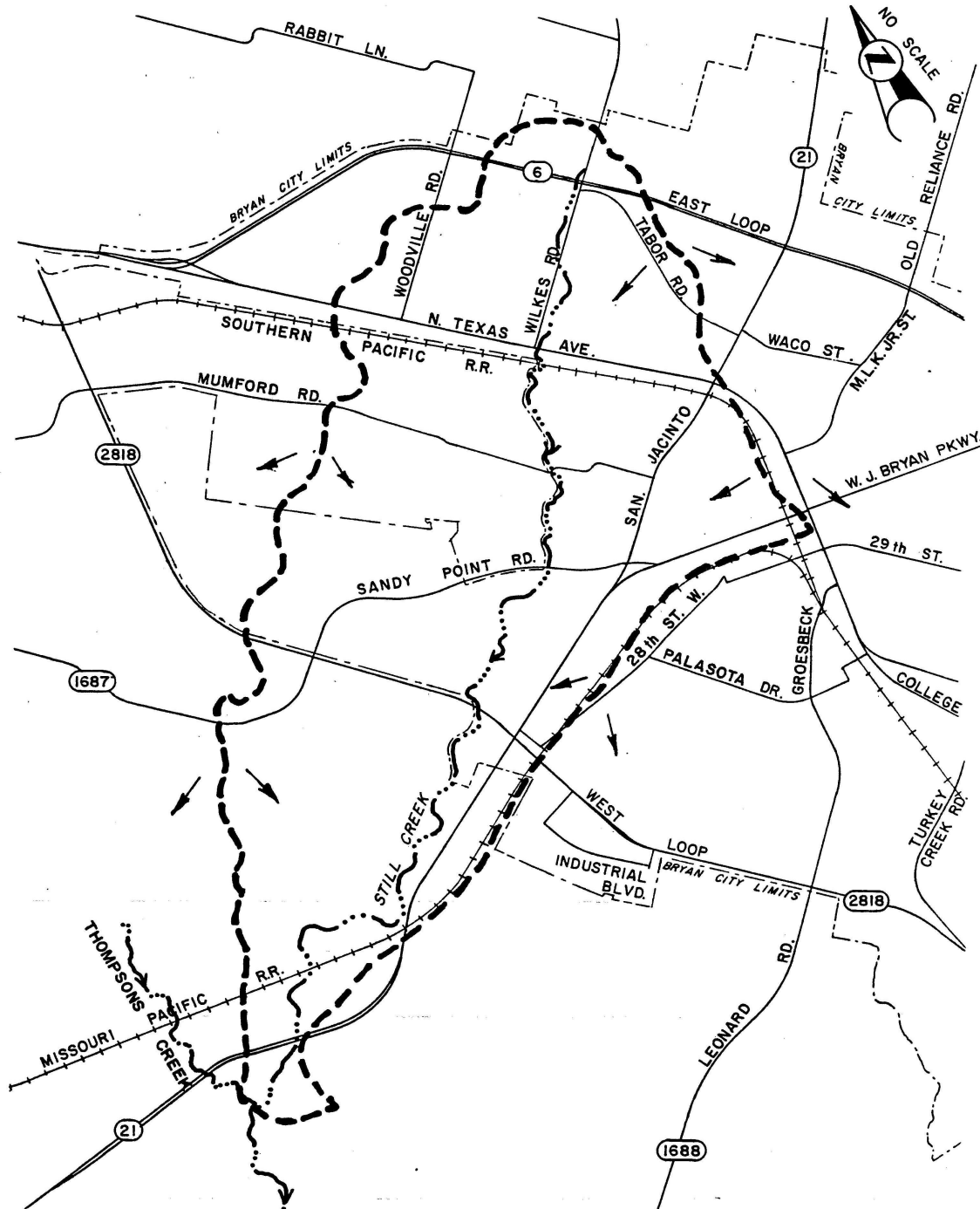
Figure 1.7
STILL CREEK DRAINAGE BASIN
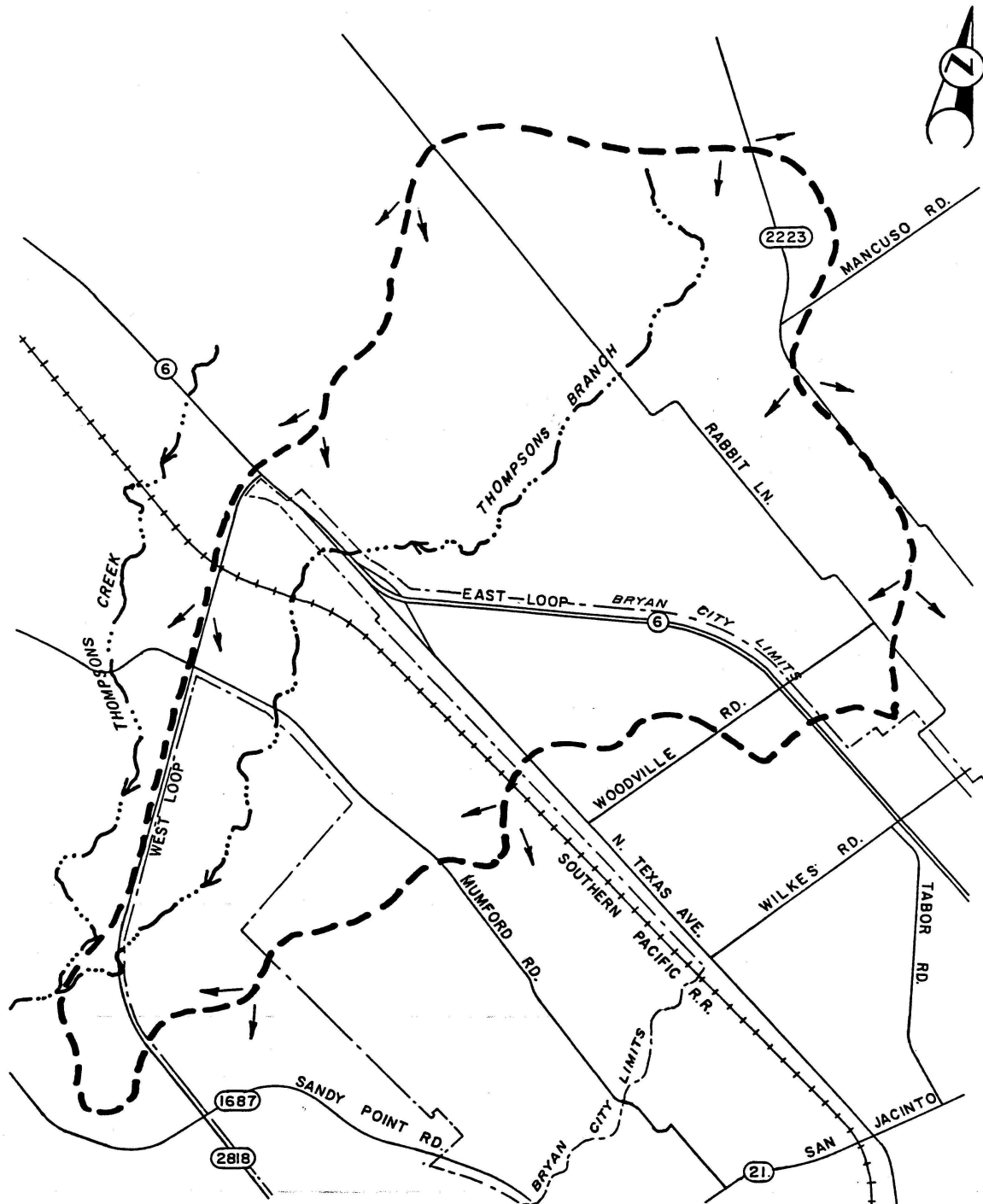
Figure 1.8

THOMPSONS BRANCH DRAINAGE BASIN
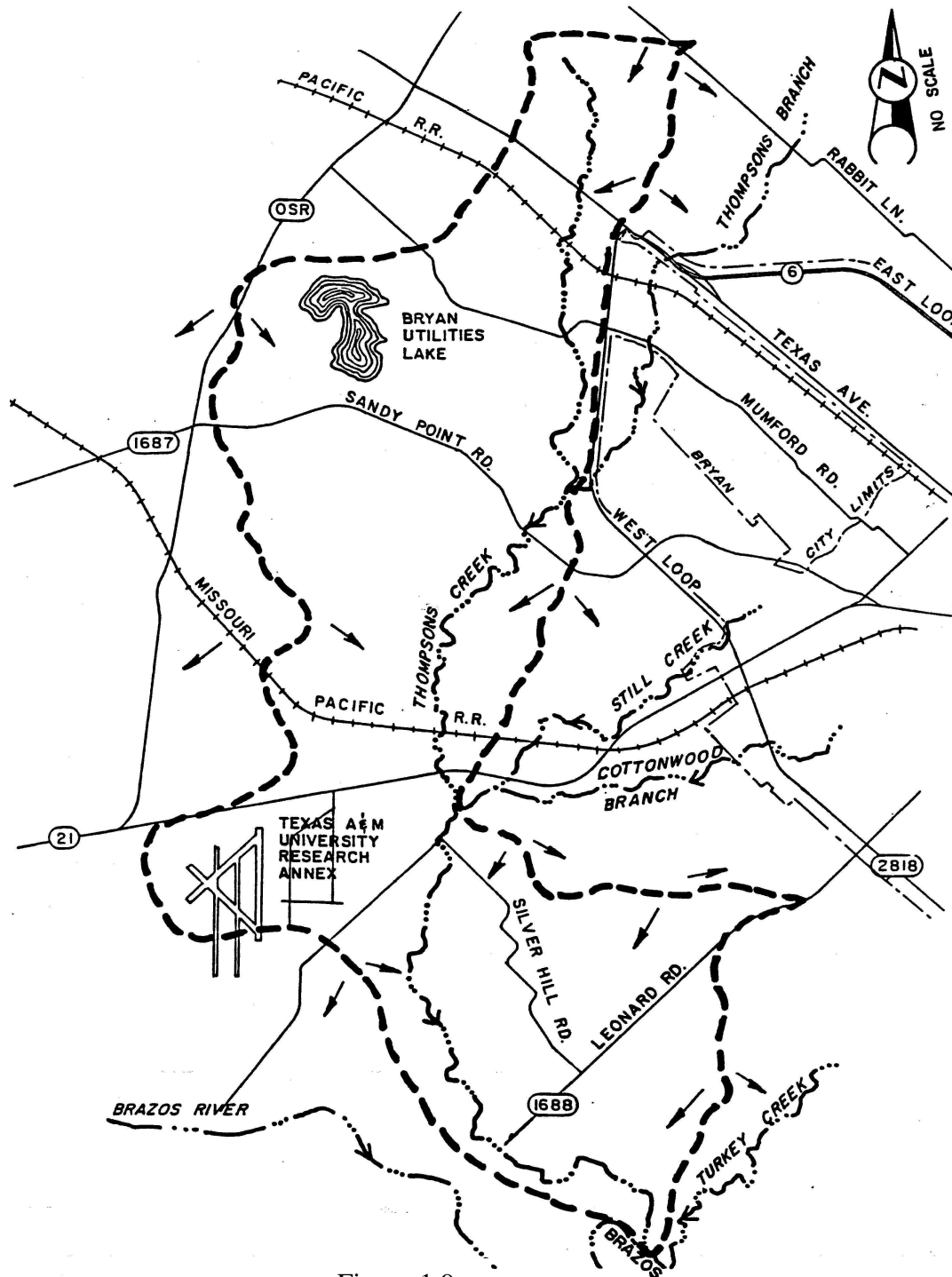
Figure 1.9
THOMPSONS CREEK DRAINAGE BASIN
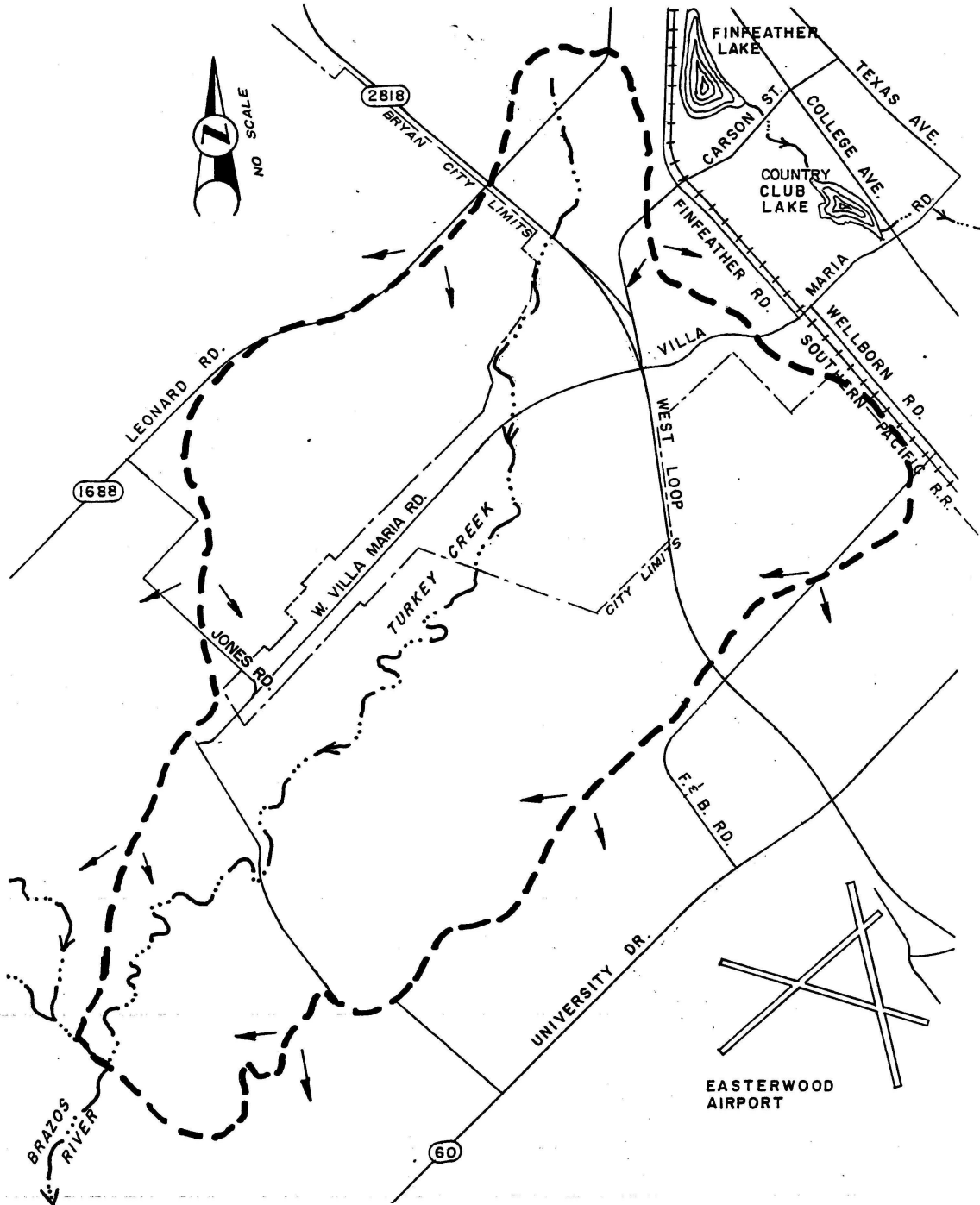
Figure 1.10

TURKEY CREEK DRAINAGE BASIN

Table 1.1 Design Storm Criteria

DESIGN STORMS AND DEVELOPMENT CONDITIONS
FOR DRAINAGE STRUCTURES AND
DETENTION/RETENTION FACILITIES

WATERSHED	NON-FLOODPLAIN DRAINAGE SYSTEMS INCLUDING CHANNELS, CULVERTS, STORM SEWERS, FLOW IN STREETS[1] ETC					
	LOCAL STREETS OR < 5 ACRES	**MINOR COLLECTORS OR 5 – 50 ACRES**	**MAJOR COLLECTORS OR 50 – 100 ACRES**	**ARTERIALS OR 100 – 500 ACRES**	**THOROUGHFARES OR > 500 ACRES**	**FACILITIES**
Briar Creek	10-Year	25-Year	50-Year	100-Year	100-Year	100-Year
Burton Creek	10-Year	25-Year	50-Year	100-Year	100-Year	100-Year
Carters Creek	10-Year	10-Year	25-Year	50-Year	100-Year	100-Year
Cottonwood Branch	10-Year	10-Year	25-Year	50-Year	100-Year	100-Year
Hudson Creek	10-Year	10-Year	25-Year	50-Year	100-Year	100-Year
Still Creek	10-Year	10-Year	25-Year	50-Year	100-Year	100-Year
Thompsons Branch	10-Year	10-Year	25-Year	50-Year	100-Year	100-Year
Thompsons Creek	10-Year	10-Year	25-Year	50-Year	100-Year	100-Year
Turkey Creek	10-Year	10-Year	25-Year	50-Year	100-Year	100-Year

Detention/Retention facilities shall be designed to limit outflows from all frequencies of storm events up to and including the listed design event to a maximum of 90% of the pre-development rates.
Facilities – Include detention facilties, bridges or other significant structures as deemed by the City Engineer.

Section Two

Hydrologic Analysis

2.1   INTRODUCTION

This section describes guidelines for different methods, which can be used to determine storm water runoff for different size watersheds within the City of Bryan. The different methods are based on watershed size. The watersheds are broken into three sizes: less than 50 acres, between 50 and 400 acres, and greater than 400 acres. Each recommended method for the corresponding watershed size is provided.

The use of the methods provided in these guidelines is preferred. However, the City Engineer will consider other methods upon submittal.

2.2   WATERSHEDS SMALLER THAN 50 ACRES

The Rational Method is one of the most frequently used methods to determine the peak runoff from a watershed. The rational method relates runoff to drainage area, surface conditions, and rainfall intensity by the following:

$$Q = C\,i\,A \tag{2.1}$$

Where
Q   =   peak runoff rate (cfs),
C   =   runoff coefficient (Table 2.1, 2.2)
i   =   average rainfall intensity (in/hr), and
A   =   drainage area (acres).

2.2.1   RUNOFF COEFFICIENT, C

The runoff coefficient, C, accounts for abstractions or losses between rainfall and runoff which may vary within a given drainage area. The type of vegetation, types of soils, and natural surface retention influence these losses. Table 2.1 provides typical values for various types of areas. Land use information may be obtained from land uses maps and aerial photographs to determine the coefficients, which are most applicable for the watershed and site visits. Table 2.1 should only be used for analysis on large watersheds, future (general) development, or for approximate purposes.

A more detailed study should develop a composite coefficient based on different surface types in the drainage area. The values given in Table 2.2 are suggested values to be used by the engineer in design.

Table 2.1 Runoff Coefficients

Description of Area	Return Period					
	2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
Park and Open Spaces						
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, > 7%	0.37	0.40	0.42	0.46	0.49	0.53
Single Family Residential Land Use						
5,000 – 7,000 sq. ft. lots						
Flat, 0-2%	0.50	0.54	0.56	0.61	0.64	0.69
Average, 2-7%	0.54	0.58	0.61	0.65	0.69	0.74
Steep, > 7%	0.56	0.60	0.63	0.68	0.71	0.76
7,000 – 10,000 sq. ft. lots						
Flat, 0-2%	0.44	0.47	0.501	0.55	0.58	0.62
Average, 2-7%	0.49	0.52	0.56	0.60	0.64	0.68
Steep, > 7%	0.52	0.55	0.58	0.63	0.66	0.71
10,000 – 20,000 sq. ft. lots						
Flat, 0-2%	0.38	0.41	0.44	0.48	0.51	0.56
Average, 2-7%	0.44	0.47	0.51	0.55	0.58	0.63
Steep, > 7%	0.47	0.51	0.54	0.58	0.62	0.66
Estate lots greater than 20,000 sq. ft.						
Flat, 0-2%	0.32	0.34	0.38	0.41	0.44	0.48
Average, 2-7%	0.38	0.41	0.44	0.49	0.52	0.56
Steep, > 7%	0.42	0.45	0.48	0.53	0.56	0.60
Multiple Family Residential Land Use						
Low Density (3 stories or less)	0.54	0.58	0.61	0.65	0.69	0.74
Medium Density (6 stories or less)	0.56	0.60	0.63	0.68	0.71	0.76
High Density (more than 6 stories)	0.59	0.63	0.66	0.71	0.75	0.80
Commercial Land Use						
Limited and General Office Building Sites	0.63	0.67	0.70	0.75	0.79	0.84
Shopping Center Sites	0.67	0.71	0.74	0.79	0.83	0.88
Neighborhood Business Districts	0.67	0.71	0.74	0.79	0.83	0.88
Office Parks	0.68	0.72	0.75	0.80	0.83	0.88
Central Business Districts	0.74	0.79	0.82	0.87	0.91	0.96
Industrial Land Use						
Limited (service station, restaurant)	0.67	0.71	0.74	0.79	0.83	0.88
General (auto sales, convenience storage)	0.67	0.71	0.74	0.79	0.83	0.88
Heavy (surface parking, warehousing)	0.74	0.79	0.82	0.87	0.91	0.96

Source: City of Temple Drainage Criteria Manual

Table 2.2 Surface Type Runoff Coefficients

Description of Surface	Return Period					
	2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
Undeveloped						
Cultivated Land						
Flat, 0-2%	0.31	0.34	0.36	0.40	0.43	0.47
Average, 2-7%	0.35	0.38	0.41	0.44	0.48	0.51
Steep, > 7%	0.39	0.42	0.44	0.48	0.51	0.54
Pasture/Range						
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, > 7%	0.37	0.40	0.42	0.46	0.49	0.53
Forest/Woodlands						
Flat, 0-2%	0.22	0.25	0.28	0.31	0.35	0.39
Average, 2-7%	0.31	0.34	0.36	0.40	0.43	0.47
Steep, > 7%	0.35	0.39	0.41	0.45	0.48	0.52
Developed						
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95
Concrete	0.75	0.80	0.83	0.88	0.92	0.97
Grass Areas (Lawns, Parks, etc.)						
Poor Condition (less than 50% coverage)						
Flat, 0-2%	0.32	0.34	0.37	0.40	0.44	0.47
Average, 2-7%	0.37	0.40	0.43	0.46	0.49	0.53
Steep, > 7%	0.40	0.43	0.45	0.49	0.52	0.55
Fair Condition (50% – 75% coverage)						
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, > 7%	0.37	0.40	0.42	0.46	0.49	0.53
Good Condition (more than 75% coverage)						
Flat, 0-2%	0.21	0.23	0.25	0.29	0.32	0.36
Average, 2-7%	0.29	0.32	0.35	0.39	0.42	0.46
Steep, > 7%	0.34	0.37	0.40	0.44	0.47	0.51

Source: Rossmiller, R.L. "The Rational Formula Revisited."
    City of Austin Drainage Criteria Manual
    City of Temple Drainage Criteria Manual

$$C_c = \frac{\sum\limits_{i=1}^{n} C_i A_i}{\sum\limits_{i=1}^{n} A_i} \tag{2.2}$$

Where

C_c	=	Composite runoff coefficient,
C_i	=	Runoff coefficient for n^{th} area, and
A_i	=	Drainage area for n^{th} area.

2.2.2   DRAINAGE AREA

The drainage area for use with the Rational Method analyses can be determined by using the City of Bryan topographical maps. When areas of the development fall outside the limits of these maps USGS Quadrangle maps can be utilized to determine the contributing drainage area of the watershed. At each location within the watershed, which the runoff is calculated, the drainage area is defined as the *total* area which contributes runoff to that location.

2.2.3   TIME OF CONCENTRATION

The time of concentration, t_c, is defined as the time required for the surface runoff to flow from the most remote point in a subbasin, subwatershed, or watershed to the computation reference point. The time of concentration is the summation of the flow time for overland or sheet flow, which occurs in headwater areas; the flow time for shallow concentrated flow, which occurs immediately downstream of overland flow; and the flow time for open channel or storm sewer flow, which tends to occur in the lower reaches of a tributary area (Walesh, 1989). The minimum allowable time of concentration for design calculations is ten (10) minutes.

Time of concentration is dependent on the type of flow whether it is overland or concentrated flow. In order to estimate the time of concentration, the watershed is broken down into different regions of known flow conditions. For each reach of the watershed the velocity is determined based on the flow conditions. For overland sheet flow, the method developed by Overton and Meadows (1976) may be used for flow distances of 300 feet or less:

$$T_t = \frac{0.007 \left(n\, L \right)^{0.8}}{\sqrt{P_i}\; S^{0.4}} \tag{2.3}$$

Where

T_t	=	travel time (hours),
n	=	Manning's roughness coefficient (Table 2.3),
L	=	Overland flow distance (feet), not to exceed 300 ft,
P_i	=	i^{th}-year recurrence interval for the 24-hr rainfall depth (inches)(Table 2.5), and
S	=	land slope (feet per foot).

This equation is based on the following assumptions:
1) Shallow uniform steady flow,
2) Constant rainfall intensity,
3) Rainfall duration of 24-hours, and
4) Infiltration does not impact travel time.

If the overland flow time is calculated to be in excess of 20 minutes, the engineer shall verify that the time is reasonable.

Source: Walesh (1989)
         Montgomery County Drainage Criteria Manual (1989)

Table 2.3 Manning's Roughness Coefficients for Sheet Flow

Description of Surface	Roughness Coefficient, n
Smooth surfaces (concrete, asphalt, gravel or bare soil)	0.011
Fallow (no residue)	0.050
Cultivated Soils	
Residue Cover (less than 20%)	0.060
Residue Cover (greater than 20%)	0.170
Grass	
Short grass prairie	0.150
Dense grass prairie	0.240
Bermuda grass	0.410
Range (natural)	0.130
Woods	
Light underbrush	0.400
Dense underbrush	0.800

Source: After U.S. Department of Agriculture (1986).

For shallow concentrated flow, the velocity may be estimated using Manning's Equation. The time of concentration can then be estimated by the following relationship.

$$T = \frac{D}{60V} \tag{2.4}$$

Where
T   =   Travel time (minutes),
D   =   Flow distance (feet), and
V   =   Average velocity of runoff (feet/sec).

2.2.4   RAINFALL INTENSITY

Rainfall intensity, i, is the average rate of rainfall in inches per hour and can be determined for the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year return periods for durations from ten minutes to 24 hours. The duration is assumed to be the time of concentration. The rainfall intensities may be determined from the intensity-duration-frequency (IDF) curves or from the equations presented in Table 2.4. These equations approximate the IDF curves within a reasonable margin of error. For the Rational Method, the critical rainfall intensity is the rainfall having a duration equal to the time of concentration of the drainage basin.

Table 2.4 Equations for Calculating Rainfall Intensities

Storm Frequency	Intensity (in/hr) i =
2-Year	$65/(t_c + 8.0)^{0.806}$
5-Year	$76/(t_c + 8.5)^{0.785}$
10-Year	$80/(t_c + 8.5)^{0.763}$
25-Year	$89/(t_c + 8.5)^{0.754}$
50-Year	$98/(t_c + 8.5)^{0.745}$
100-Year	$96/(t_c + 8.0)^{0.730}$

after: TxDOT Hydraulic Manual, 1986.

2.2.5   HYDROGRAPH DEVELOPMENT

The development of the runoff hydrograph is accomplished through the use of a triangular shape hydrograph. The peak of the hydrograph is set equal to the peak discharge for pre- and post development runoff rates as determined by the Rational Method. The peak runoff rate occurs at a time equal to the time of concentration and the total duration of the runoff hydrograph is equal to 3 times the time of concentration. Figure 2.1 shows a generalized layout of the storm hydrograph. The larger triangle represents the post development runoff hydrograph and the smaller triangle shaded gray is the pre-development runoff hydrograph. The storage volume required for detention is equal to the difference between the two triangular hydrographs and is represented by the hatched area shown in Figure 2.1. The may vary between the pre and post (as in the example in Section 2.2.6), however, small sites will often both be the minimum t_c of 10 minutes.
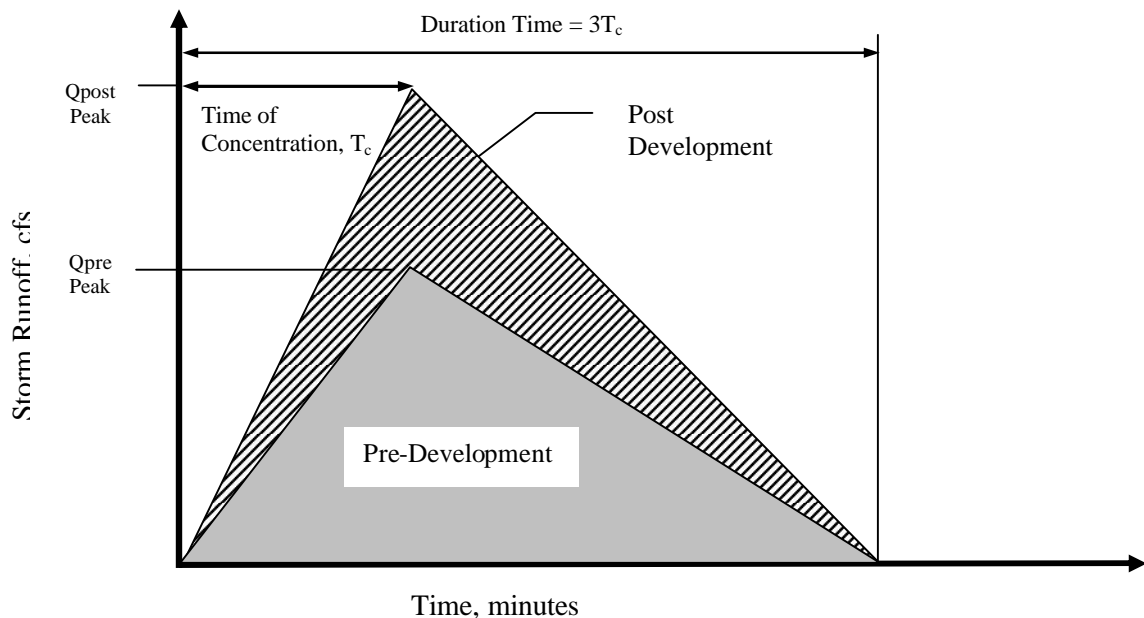
Figure 2.1 Runoff Hydrographs

2.2.6 RATIONAL METHOD AND HYDROGRAPH EXAMPLE

Given: A piece of land (3.8 acres) in north Bryan has been purchased for residential development. Currently the property is pastureland. The planned development for the land is SF-5 residential. From contours obtained from the City of Bryan, the site has an average slope of 1.5%. Due to the location of the detention pond it has been determined that the time of concentration to the pond is 15 minutes pre-development and 10 minutes post development.

Required: Determine the maximum post development runoff rate and the storage volume required for a detention pond.

Step 1: Calculate pre-development runoff rate using Equation 2.1. Since runoff is calculated to establish the volume for a detention pond the Return Period used is 100-yr.

$$Q = CiA$$

C - Table 2.2 for pastureland and 1.5% slope, i – Table 2.4 w/ T_c = 15 minutes, A – Given.

$$Q_{pre} = (0.41)\ (9.73\ \text{in/hr})\ (3.8\ \text{acres})$$
$$Q_{pre} = 15.2\ \text{cfs}$$
$$Q_{pre\text{-}allowed} = 15.2\ \text{cfs} \times 0.90\ (\text{Table 1.1}) = 13.68\ \text{cfs}$$

Step 2: Calculate post development runoff rate as in Step 1.

$$Q_{post} = (0.69) (11.64 \text{ in/hr}) (3.8 \text{ acres})$$
$$Q_{post} = 30.5 \text{ cfs}$$

Step 3: Calculate volume of triangular runoff hydrographs.

$$Vol_{pre} = \frac{1}{2} (Qpre) (3T_c) (60)$$
$$Vol_{pre} = \frac{1}{2} (13.68 \text{ cfs}) (45) (60)$$
$$Vol_{pre} = 18,468 \text{ cubic feet}$$

$$Vol_{post} = \frac{1}{2} (Q_{post}) (3T_c) (60)$$
$$Vol_{post} = \frac{1}{2} (30.5) (30) (60)$$
$$Vol_{post} = 27,450 \text{ cubic feet}$$

$$\text{Volume required} = Vol_{post} - Vol_{pre}$$
$$\text{Volume Required} = 27,450 \text{ ft}^3 - 18,468 \text{ ft}^3$$
$$\text{Volume Required} = 8,982 \text{ ft}^3$$

Therefore the maximum release rate from the detention pond is 13.68 cfs (90% of the Q_{pre}) and the required storage volume of water is preliminarily approximated at 8,982 cubic feet for the 100-year recurrence interval. A routing analysis is required for final pond sizing in accordance with 5.2.3. This routing analysis should analyze the outlet structure for each event. The final constructed volume of the pond will be required to be larger than the volume of water calculated to account for the free board required as outlined in Section 5 *Detention Ponds*.

2.2.7 SUBMITTAL REQUIREMENTS (FOR WATERSHEDS SMALLER THAN 50 ACRES)

The following list serves as a checklist for items, which are required to be included in a drainage report submitted for approval.

1. Project Scope – should include project location, project size, and proposed development
2. Pre-Development Drainage Patterns and Major Drainage Basin Information
3. Drainage Design Criteria
4. Stormwater Runoff computations
      a. Runoff Coefficients
      b. Time of Concentration with drainage path highlighted on a drawing
      c. Rainfall Intensity
      d. Area of Site
      e. Stormwater Runoff Quantities
5. Show maps with Pre- and Post Development Drainage areas clearly identified with arrows indicating direction of flow
6. Topographic maps shall have contours shown at 1-ft intervals

2.3 WATERSHEDS LARGER THAN 50 AND SMALLER THAN 400 ACRES

The Soil Conservation Service (SCS) Technical Release No. 55 (TR55) method calculates peak discharges and volumes for different recurrence intervals and is recommended for determining runoff for watersheds larger than 50 acres and smaller than 400 acres. The TR55 method can be used to develop runoff hydrographs for sub-basins and combine them through the watershed system. The TR55 method is applicable to rural and urban watersheds under the following conditions:

1. Use of one to four 24-hour rainfall hyetographs.
2. Average antecedent moisture conditions are defined as a total of 1.4 to 2.1 in. of rainfall during the 5-day period immediately preceding the design rainfall. Adjustments may be made to simulate wet or dry antecedent moisture conditions.
3. All flow is assumed to be sheet flow, shallow concentrated flow, or open channel flow. Surcharged pipe flow and flow on street surfaces are not taken into account.

The SCS method is an acceptable and recommended method to analyze a watershed between 50 and 400 acres within the City of Bryan. For more information and guidance on the TR-55 method, the engineer is referred to the following publications, which discuss the methodology in more detail:

> TR-55: Urban Hydrology for Small Watersheds (available from Dept. of Agricultural via the Web: http://www.wcc.nrcs.usda.gov/water/quality/common/tr55/tr55.html)
> Urban Surface Water Management, by Stuart G. Walesh, John Wiley & Sons, 1989
> TR-149: A Method for Estimating Volume and Rate of Runoff in Small Watersheds
> TR-20: Computer Program for Project Formulation, Hydrology

The TR-55 manual provides numerous examples and explanations on the limitations of the method and it applicability. The worksheets used in conjunction with the SCS method are given in Figures 2.2 through 2.9.

2.3.1 SUBMITTAL REQUIREMENTS (FOR WATERSHEDS LARGER THAN 50 AND SMALLER THAN 400 ACRES)

The following list serves as a checklist for items, which are required to be included in a drainage report submitted for approval.

1. Stormwater Runoff computations
   a. SCS Curve Numbers
   b. Time of Concentration with drainage path highlighted on a drawing
   c. Area of Site
   d. Stormwater Runoff Quantities – Graphical or Tabular Method
   e. TR55 worksheets used in the SCS method (Figures 2.2-2.9)

2. Any Computer Program generated output shall be submitted in an Appendix and shall not constitute a report by itself
3. Any and all computation equations
4. Show maps with Pre- and Post Development Drainage areas clearly identified with arrows indicating direction of flow
5. Topographic maps shall have contours shown at 2-ft intervals

If the engineer wishes to utilize the accompanying computer package to the TR-55 method, a 3.5" disk or CD shall accompany the drainage report as part of the submittal. The disk or CD shall contain all necessary input parameters. The disk or CD shall be clearly labeled with project information and method of analysis.

2.4 WATERSHEDS LARGER THAN 400 ACRES

The hydrological analysis of watersheds larger than 400 acres usually entails the use of some type of hydrological model. The hydrological model provides a more complex approach to computing runoff for large areas. These models are also useful if storage is to be analyzed or if complex hydrological conditions exist within the watershed.

Hydrologic analyses of watersheds larger than 400 acres shall be done using either HEC-1 or HEC-HMS computer program. These programs were developed by the U.S. Army Corps of Engineers and automatically distribute the rainfall over a 24-hour period such that the maximum rainfall intensity falls within the middle of the storm event. The program simulates the response of a drainage basin through unit hydrograph procedures, loss rate functions, and channel and reservoir routing options. Several elements required to develop a computer model are: Precipitation Data, Precipitation Loss Computations, Development of Unit Hydrograph, Adjustments for Ponding, and Streamflow Routing.

2.4.1 PRECIPITATION DATA

Precipitation data is input in terms of hyetographs taken from historical or synthetic storms. Precipitation data to be used for the City of Bryan is shown in Table 2.5. The precipitation data can be entered on the PH card in HEC-1.

Table 2.5 Depth-Duration-Frequency Data for Bryan, Texas (TP-40 and Hydro 35)

Recurrence Interval	Rainfall Depth for Given Duration (inches)								
	5-min	15-min	30-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
2-year	0.53	1.15	1.68	2.20	2.60	2.86	3.33	3.80	4.50
5-year	0.60	1.33	2.00	2.68	3.36	3.70	4.41	5.25	6.20
10-year	0.66	1.46	2.24	3.02	3.94	4.41	5.29	6.28	7.40
25-year	0.75	1.66	2.59	3.52	4.57	5.14	6.20	7.42	8.40
50-year	0.82	1.82	2.87	3.91	5.10	5.65	6.95	8.45	9.80
100-year	0.89	1.98	3.14	4.30	5.60	6.30	7.90	9.50	11.00

2.4.2   PRECIPITATION LOSS COMPUTATIONS

Infiltration losses will be accounted for using the SCS Curve Number method. Equations 2.5 and 2.6 are used to compute the runoff for a given rainfall event.

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \tag{2.5}$$

$$S = \frac{1000}{CN} - 10 \tag{2.6}$$

Where
Q	=	Total runoff (inches),
P	=	Total rainfall (inches)(Table 2.5),
S	=	Amount of rainfall infiltration prior to runoff (inches), and
CN	=	SCS Curve Number.

The Curve Number is a function of soil cover, land use type, and antecedent moisture conditions and is given in Table 2.6. The SCS soil classification uses four soil classification systems as follows: Group A: deep sand, deep loess, aggregated silts; Group B: shallow loess, sandy loam; Group C: clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay; Group D: soils that swell significantly when wet, heavy plastic clays, and certain saline soils. Usually the best source for determining the soil group for a particular drainage basin is the *Soil Survey for Brazos County, Texas (SCS, 1946)*.

For watersheds with varying land uses and soil types, it may be necessary to compute a composite SCS curve number. The composite SCS curve number may be computed using the Equation 2.7.

$$CN_c = \frac{\sum_{i=1}^{n} CN_i A_i}{\sum_{i=1}^{n} A_i} \tag{2.7}$$

Where
CN_c	=	SCS composite curve number,
CN_i	=	SCS curve numbers for different land uses and soil types, and
A_i	=	Drainage areas corresponding to CN_i values.

The SCS has developed three antecedent moisture conditions, which have significant effect on runoff potential. They are as follows: Condition I: soils are dry but not to wilting point, Condition II: average conditions, and Condition III: heavy rainfall or light rainfall with low temperatures have occurred within past 5 days; saturated soils.

Table 2.6 SCS Runoff Curve Numbers

Cover Description	Curve Numbers for Hydrologic Soil Group[a]			
Cover Type and Hydrologic Conditions	**Average Percent Impervious Area[b]**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established					
Open space (lawns, parks, golf courses, cemeteries, etc.)[c]:					
Poor condition (grass cover < 50%)……………………		68	79	86	89
Fair condition (grass cover 50 to 70%)…………………		49	69	79	84
Good condition (grass cover > 75%)…………………...		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding					
right-of-way)………………………………………....		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved, open ditches (including right-of-way)……………		83	89	92	93
Gravel (including right-of-way)……………………….……		76	85	89	91
Dirt (including right-of-way)……………………….……		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]……....		63	77	85	88
Artificial desert landscaping (impervious weed barrier,					
desert shrub with 1 to 2-inch sand and gravel mulch and					
basin borders)………………………………………....		96	96	96	96
Urban Districts:					
Commercial and business	85	89	92	94	95
Industrial	72	91	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)…………………………....	65	77	85	90	92
1/4 acre…………………………………………………..	38	61	75	83	87
1/3 acre…………………………………………………..	30	57	72	81	86
1/2 acre…………………………………………………..	25	54	70	80	85
1 acre……………………………………………………	20	51	68	79	84
2 acres…………………………………………………....	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[e]		77	86	91	94

Cover Description		Curve Numbers for Hydrologic Soil Group[a]			
Cover Type and Hydrologic Conditions	Average Percent Impervious Area[b]	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing.[f,h]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay	--	30	58	71	78
Brush—brush-weeds-grass mixture with brush the major elements.[g,h]	Poor	48	67	77	
	Fair	35	56	70	
	Good	30	48	65	
Woods—grass combination (orchard or tree farm)[I]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[h,j]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots	---	59	74	82	86

a   Average runoff condition, and $I_a = 0.2S$

b   The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: Impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using Figures 2.3 or 2.4 in TR-55 publication.

c   CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

d   Composite CN's for natural desert landscaping should be computed using Figures 2.3 or 2.4 located in TR-55 based on the impervious area percentage (CN=98) and the pervious area CN. The pervious are CN's are assumed equivalent to desert shrub in poor hydrologic condition.

e   Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figures 2.3 or 2.4, located in TR-55 based on the degrees of development (impervious area percentage) and the CN's for the newly graded pervious area.

f   Poor: < 50% ground cover or heavily grazed with no mulch.
    Fair: 50 to 75% ground cover and not heavily grazed
    Good: > 75% ground cover and lightly or only occasionally grazed.

g   Poor: < 50% ground cover.
    Fair: 50 to 75% ground cover.
    Good: > 75% ground cover.

h   Actual curve number is less than 30; use CN = 30 for runoff computations.

i   CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pastures.

J   Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
    Fair: Woods are grazed but not burned, and some forest litter covers the soil.
    Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Source: Natural Resource Conservation Service, Urban Hydrology for Small Watersheds, Technical Release No. 55. June 1986.

2.4.3   UNIT HYDROGRAPH DEVELOPMENT

Once the excess rainfall from the design storm has been established, it is necessary to determine the runoff hydrograph. The SCS unit hydrograph method for a watershed shall be used in this analysis.   The SCS method uses three parameters to define a unit hydrograph: time of concentration (T_c), storage coefficient (R), and a time area curve.   The storage coefficient indicates what storage volume is available within a given watershed. The value of the coefficient is directly related to the available storage in that watershed. The time area curve gives the total area of the watershed, which contributes runoff to the point of analysis as a function of time. The HEC-1 and HEC-HMS programs provide some standard curves to estimate the time area curve based on watershed geometry.

2.4.4   STREAMFLOW ROUTING

HEC-1 and HEC-HMS programs have a number of routing methods. Two routing methods are recommended for use within the City of Bryan is the Modified Puls. (See Section 3.5.7.2)

2.4.5   COMBINING HYDROGRAPHS

In large watersheds it may be necessary to combine hydrographs in order to accurately depict the runoff with one hydrograph where two or more sub-basins intersect and combine flows. Should this occur the drainage report shall explain the location of these intersections and provide the necessary input files in conjunction with the report.

2.4.6   SUBMITTAL REQUIREMENTS FOR WATERSHEDS LARGER THAN 400 ACRES

The following is a list of items, which need to be included into a drainage report, which covers watersheds of more than 400 acres.

1. Stormwater Runoff computations
        a. Time of Concentration with drainage path highlighted on a drawing
        b. Area of Site
2. All Computer Program Input Parameters in Tabular Form
3. Any Computer Program generated output shall be submitted in an Appendix and shall not constitute a report by itself
4. Any and all computation equations
5. Show maps with Pre- and Post Development Drainage areas clearly identified with arrows indicating direction of flow
6. In addition to the above items contained within the report, a 3.5" floppy disk or CD with all input data shall accompany the report. The disk or CD shall be clearly labeled with project information and method of analysis.

Worksheet 2: Runoff curve number and runoff

Project	By	Date
Location	Checked	Date

Check one: ☐ Present ☐ Developed

1. Runoff curve number

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN [1/] Table 2-2	Figure 2-3	Figure 2-4	Area ☐ acres ☐ mi² ☐ %	Product of CN x area

[1/] Use only one CN source per line

Totals ➡

$$\text{CN (weighted)} = \frac{\text{total product}}{\text{total area}} = \underline{\hspace{2cm}} = \underline{\hspace{1cm}} \quad ; \quad \text{Use CN} \blacktriangleright \boxed{}$$

2. Runoff

	Storm #1	Storm #2	Storm #3
Frequency .... yr			
Rainfall, P (24-hour) .... in			
Runoff, Q .... in			

(Use P and CN with table 2-1, figure 2-1, or equations 2-3 and 2-4)

Figure 2.2 SCS Worksheet Number 2

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project		By		Date
Location		Checked		Date

Check one: ☐ Present ☐ Developed

Check one: ☐ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to Tc only)

	Segment ID	
1. Surface description (table 3-1) ....
2. Manning's roughness coefficient, n (table 3-1) ....
3. Flow length, L (total L † 300 ft) .... ft
4. Two-year 24-hour rainfall, P_2 .... in
5. Land slope, s .... ft/ft
6. $T_t = \dfrac{0.007 \ (nL)^{0.8}}{P_2^{\ 0.5} s^{0.4}}$ Compute T_t .... hr

☐ + ☐ = ☐

Shallow concentrated flow

	Segment ID	
7. Surface description (paved or unpaved) ....
8. Flow length, L ....ft
9. Watercourse slope, s .... ft/ft
10. Average velocity, V (figure 3-1) .... ft/s
11. $T_t = \dfrac{L}{3600 \ V}$ Compute T_t .... hr

☐ + ☐ = ☐

Channel flow

	Segment ID	
12. Cross sectional flow area, a .... ft²
13. Wetted perimeter, p_w .... ft
14. Hydraulic radius, $r = \dfrac{a}{p_w}$ Compute r .... ft
15 Channel slope, s .... ft/ft
16. Manning's roughness coefficient, n ....
17. $V = \dfrac{1.49 \ r^{2/3} \ s^{1/2}}{n}$ Compute V ....ft/s
18. Flow length, L .... ft
19. $T_t = \dfrac{L}{3600 \ V}$ Compute T_t .... hr

☐ + ☐ = ☐

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... Hr ☐

D–3

Figure 2.3 SCS Worksheet Number 3

Average velocities for estimating travel time for shallow concentrated flow
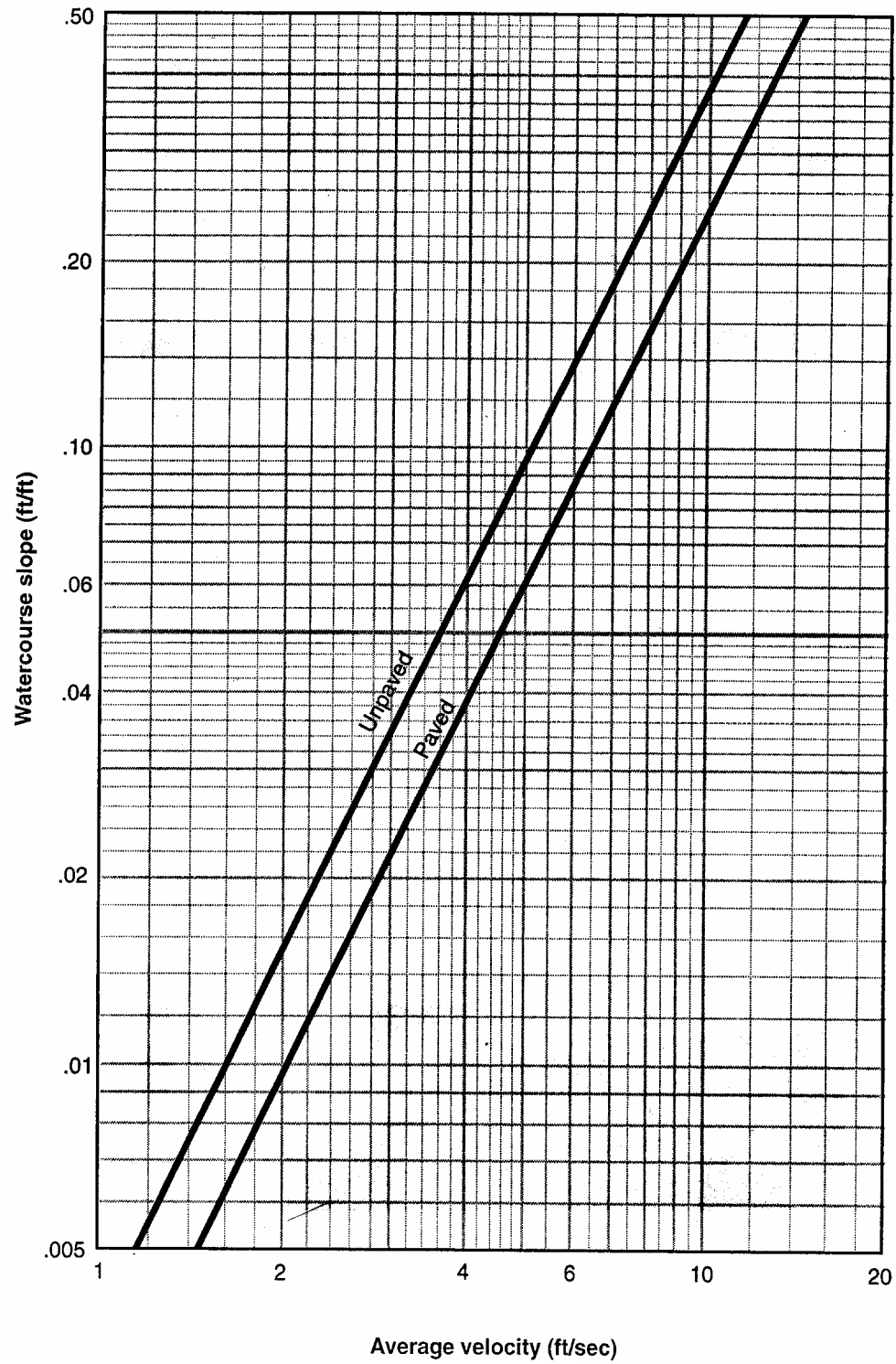
Figure 2.4 Average Velocity of Overland Flow

Worksheet 4: Graphical Peak Discharge method

Project	By	Date
Location	Checked	Date

Check one: ☐ Present   ☐ Developed

1. Data

Drainage area .... $A_m =$ _____ mi^2 (acres/640)

Runoff curve number .... CN = _____ (From worksheet 2)

Time of concentration .... $T_c =$ _____ hr (From worksheet 3)

Rainfall distribution .... = _____ (I, IA, II III)

Pond and swamp areas sprea
throughout watershed .... = _____ percent of A_m (_____ acres or mi^2 covered)

	Storm #1	Storm #2	Storm #3
2. Frequency .... yr			
3. Rainfall, P (24-hour) .... in			
4. Initial abstraction, I_a .... in (Use CN with table 4-1)			
5. Compute I_a / P			
6. Unit peak discharge, q_u .... csm/in (Use T_c and I_a / P with exhibit 4– _____)			
7. Runoff, Q .... in (From worksheet 2) Figure 2-6			
8. Pond and swamp adjustment factor, F_p .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond ans swamp area.)			
9. Peak discharge, q_p .... ft^3/s			

(Where $q_p = q_u A_m Q F_p$)

(210-VI-TR-55, Second Ed., June 1986)

Figure 2.5 SCS Worksheet Number 4

Worksheet 5a: Basic watershed data

Subarea name	Drainage area A_m (mi²)	Time of concentration T_c (hr)	Travel time through subarea T_t (hr)	Downstream subarea names	Travel time summation to outlet ΣT_t (hr)	24-hr rain-fall P (in)	Runoff curve number CN	Runoff Q (in)	A_mQ (mi²—in)	Initial abstraction I_a (in)	I_a/P

Project _____   Location _____   By _____   Date _____

Check one: ☐ Present ☐ Developed   Frequency (yr) _____   Checked _____   Date _____

From worksheet 3   From worksheet 2   From table 5-1

Figure 2.6 SCS Worksheet Number 5a

Worksheet 5b: Basic watershed data

| Project | | | Location | | By | Date |
| Check one: ☐ Present ☐ Developed | | | | Frequency (yr) | Checked | Date |

Basic watershed data used 1/

Select and enter hydrograph times in hours from exhibit 5-II 2/

Discharges at selected hydrograph times 3/ (cfs)

Subarea name	Subarea T_c (hr)	ΣT_t to outlet (hr)	I_a/P	$A_m Q$ (mi²–in)												
Composite hydrograph at outlet																

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is $A_m Q$ multiplied by tabular discharge from appropriate exhibit 5.

D–6   (210-VI-TR-55, Second Ed., June 1986)

Figure 2.7 SCS Worksheet Number 5b

Worksheet 6a: Detention basin storage, peak outflow discharge (q_o) known

Project	By	Date
Location	Checked	Date

Check one: ☐ Present   ☐ Developed

☐ Elevation or ☐ stage

Detention basin storage (acre feet)

1. Data:
   Drainage area .... A_m = _____ mi²
   Rainfall distribution
   type (I, IA, II, III)   = _____

	1st Stage	2nd Stage
2. Frequency .... yr		
3. Peak inflow discharge q_i .... ft³/s (from worksheet 4 or 5b)		
4. Peak outflow discharge q_u .... ft³/s		[1/]
5. Compute $\dfrac{q_o}{q_i}$		

6. $\dfrac{V_s}{V_r}$ .... [|]
   (Use $\dfrac{q_o}{q_i}$ with figure 6-1)

7. Runoff, Q .... in [|]
   (From worksheet 2)

8. Runoff volume
   V_r .... ac ft [|]
   ($V_r = QA_m\ 53.33$)

9. Storage volume,
   V_s .... ac-ft [|]

   $(V_s = V_r\ (\dfrac{V_s}{V_r}))$

10. Maximum storage E_{max} [|]
    (from plot)

1/ 2nd stage q_o includes 1st stage q_o.

Figure 2.8 SCS Worksheet Number 6

Worksheet 6b: Detention basin storage, storage volume (V_s) known

Project	By	Date
Location	Checked	Date

Check one: ☐ Present ☐ Developed

☐ Elevation or ☐ stage

Detention basin storage

1. Data:
   Drainage area .... A_m = _____ mi²
   Rainfall distribution
   type (I, IA, II, III) = _____

	1st Stage	2nd Stage
2. Frequency .... yr		
3. Storage volume V_s .... ac-ft		
4. Runoff, Q .... in (from worksheet 2)		
5. Runoff volume ..... ac-ft (V_r = QA$_m$ 53.33)		

6. Compute $\dfrac{V_s}{V_r}$ ....

7. $\dfrac{q_o}{q_i}$ .... in
   (Use $\dfrac{V_s}{V_r}$ with figure 6-1)

8. Peak inflow discharge
   q_i .... in
   (From worksheet 4 or 5b)

9. Peak outflow discharge   1/
   q_o .... ft³/s

   $(q_o = q_i \left(\dfrac{q_o}{q_i}\right))$

10. Maximum storage E_{max}
    (from plot)

1/ 2nd stage q_o includes 1st stage q_o.

Figure 2.9 SCS Worksheet Number 6B
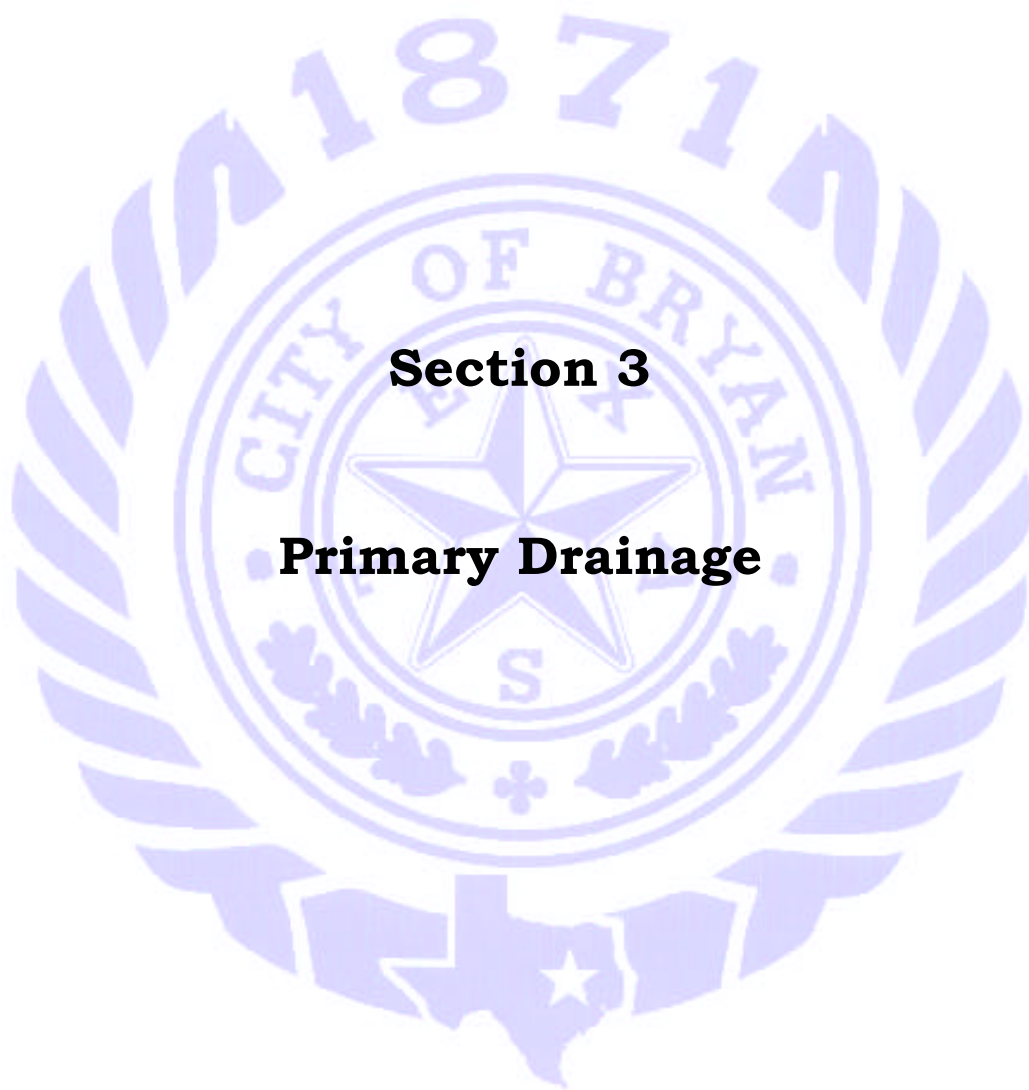
Section 3

Primary Drainage

3.1 INTRODUCTION

This section describes the design guidelines for development involving Primary Drainage Facilities. The different hydraulic analyses for Primary Drainage broadly include the following subsections *Culverts, Bridges,* and *Open Channels*.

Beyond approval through the City of Bryan Engineering Services, proposed improvements involving Primary Drainage often require the approval of the Federal Emergency Management Agency (FEMA) and the U.S. Army Corps of Engineers, *Floodplains* and *U.S. Waters,* are dedicated to these related approval processes.

3.2 CULVERTS

3.2.1 GENERAL

3.2.1.1 Structural Requirements

Gasketed, such as O-Ring or Profile Gasket in accordance with ASTM C443, Reinforced Concrete Pipe (RCP) should be a minimum of ASTM C-76 Class III for designs with a minimum one foot of cover or ASTM C-76 Class IV for less than one foot of cover or if cover is less than 1/8 of the pipe diameter. The O-Ring or Profile Gasket RCP is required under pavement. Where not under pavement, Tongue and Groove RCP, with mastic, is allowed. Upon approval by the City Engineer, HDPE pipe will be allowed as a special use for an areas that are difficult to access.

Reinforced Concrete Box Culvert (RCBC) should be a minimum of ASTM C-789 for minimum of 2 feet of cover or ASTM C-850 for less than 2 feet of cover. Cast in place RCBC shall meet or exceed TxDOT design standards.

Corrugated Metal Pipes (CMP) are not allowed for public storm sewer. The City Engineer may approve special and unique uses for plastic pipes on a special case by case basis.

3.2.1.2 Culvert Sizes

RCP: The minimum size shall be 18-inch diameter.

RCBC: The minimum size shall be 2' x 2'.

3.2.1.3 Temporary Culverts

Temporary Culverts may be utilized on sites during construction. The culverts should be designed for minimum of 2-year storm and be constructed in such a way to provide a

low-flow spillway. CMP and plastic pipes are acceptable. Temporary culverts must be removed at completion of the project. A spillway for a 10-year storm should be designed and constructed.

3.2.2 CULVERT LAYOUT GEOMETRY

3.2.2.1 Street Crossing

Culvert crossing alignments should match the centerline alignment of the stream to avoid erosion. Longitudinal runs shall be avoided. The culvert shall extend a minimum of 15 feet beyond the edge of pavement. This extension may require additional ROW.

3.2.2.2 Roadside / Driveway Culvert

Culverts in roadside ditches or driveway culverts shall be placed within the public ROW, parallel to the street, and aligned with the designed flow-line.

3.2.2.3 Side Slopes

The maximum side slopes for all grading in the vicinity of culvert headwall shall be 4:1 with a desirable slope of 6:1.

3.2.2.4 Transitions/Deflections

Joint deflections are not allowed. Realignments shall be accommodated by means of junction boxes. Culvert transitions in size shall match soffits at a minimum and shall only occur at a junction box.

3.2.2.5 Culvert Easements

Easements, whether involving culverts or open channel, shall be required on a parcel where concentrated runoff from an adjoining upstream parcel traverses the subject parcel. Culverts should typically be located within a street ROW. When outside a ROW, minimum easement widths for culverts are depicted below.

Table 3.1 Easement Widths for Culverts

Culvert Size	or	Culvert Cover	Minimum Easement Width
18 inch		4 feet	20 feet
42 inch		6 feet	25 feet
72 inch		8 feet	30 feet
>60 inch		>8 feet	to be determined

A minimum of a 15-foot Access Easement is required for maintenance access.

3.2.3 CULVERT DESIGN

3.2.3.1 Introduction

Several procedures utilizing empirical formulas, charts or nomographs have been used to design culverts. Acceptable methods can be found in the 2001 TxDOT Hydraulic Manual and the Bureau of Public Roads' Hydraulic Engineering Circular No. 5, "Hydraulic Charts for the Selection of Highway Culverts" and Circular No. 10, "Capacity Charts for the Hydraulic Design of Highway Culverts." The American Concrete Pipe Association's website www.concrete-pipe.org/tech.html#design provides associated charts, nomographs, and concise culvert design procedures in Chapter 3, *Hydraulics of Culverts*. Software packages such as Haestad Method's *Culvert Master* may also be utilized. All culverts under streets, driveways, on Site Plans, as well as temporary culverts, shall utilized the following design methods.

3.2.3.2 Design Storm

Refer to Table 1.1 – Design Storm Criteria in Section 1.

3.2.3.3 Inlet Control

Inlet Control exists when the flow allowed at the upstream entrance is less than the flow permitted through the barrel of the culvert. Therefore factors influencing the barrel rate such as slope, length and surface roughness do not control flow. Under Inlet Control, culverts will be partially full.

3.2.3.4 Outlet Control

Outlet Control exists when the flow permitted through the barrel is less than the flow allowed at the upstream entrance to the culvert. Outlet Control may should be referred to as barrel control in that the control factors include shape, slope, length and surface roughness, tailwater depth, headwater depth and inlet geometry. Under Outlet Control, culverts may be full or partially full.

3.2.3.5 Headwaters

The headwaters on channels upstream of a roadway or driveway shall be the lesser of 1 foot below the top of pavement edge or 2 feet below an adjacent structure's finish floor elevation.

3.2.3.6 Tailwaters

The tailwaters on channels downstream of a roadway or driveway shall at a minimum be set at the downstream soffit elevation.
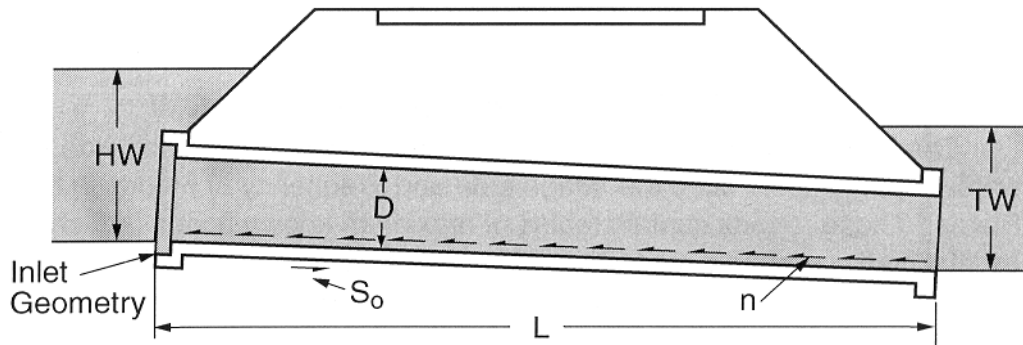

Figure 3.1 Factors Influencing Culvert Discharge

Where:
  D = inside diameter for circular pipe (ft.)
  HW = headwater depth at culvert entrance (ft.)
  L = length of culvert (ft.)
  n = Manning's surface roughness (dimensionless)
  S_o = slope of the culvert pipe (ft./ft.)
  TW = tailwater depth at the culvert outlet (ft.)
  K_e = Entrance Loss (dimensionless)

3.2.3.7 Manning's 'n' Values for Culverts

Table 3.2 Manning's 'n' Values for Culverts

Culvert Material	n
Plastic, glass, drawn tubing	0.009
Concrete pipe, asphalted cast iron	0.013*
Cast-iron pipe	0.015
Corrugated metal pipe	0.022

Source: Water Resources and Environmental Engineering
(* n=0.012 may be utilized as in the Nomograph Method)

3.2.3.8 'Ke' Entrance Loss Coefficients

Table 3.3 *'Ke' Entrance Loss Coefficients*

Inlet or Headwall Description	Ke
Concrete Pipe	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^o or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Corrugated Metal Pipe	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^o or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Reinforced Concrete Box	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 sides	0.5
Rounded 3 edges to radius of 1/12 barrel dimension, or	
beveled edges 3 sides	0.2
Wingwalls at 30° or 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension,	
or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or
concrete, are the sections commonly available from manufacturers. From
limited hydraulic tests they are equivalent in operation to a headwall in both
<u>inlet</u> and <u>outlet</u> control. Some end sections, incorporating a <u>closed</u> taper in
their design has a superior hydraulic performance.
Source: <u>Haestad Method's Culvert Master</u>

3.2.4 CULVERT EXAMPLE PROBLEMS

3.2.4.1 Example 1 – Culvert Software

There are many acceptable and user-friendly software packages available to design or analyze a culvert. The window pasted below depicts an example of such input and output. Note, that the Chart Procedure in Examples 2 and 3 evaluate the same input and arrives and the same culvert sizing and velocity.

List Design Data:
A.   Q50 = 225 cubic feet per second
B.   L = 200 feet
C.   So = 0.01 feet per foot
D.   Allowable HW = 10 feet for 50-year storm
E.   TW = 4.0 feet for 50-year storm
F.   Circular concrete culvert with a projecting entrance, n = 0.012

Culvert Calculator - Example - 1

Solve For: Size

Culvert
Discharge: 225.00 cfs
Maximum Allowable HW: 110.00 ft
Tailwater Elevation: 101.50 ft

Section
Shape: Circular
Material: Concrete
Size: 54 inch
Number: 1
Mannings: 0.012

Inlet
Entrance: Groove end projecting
Ke: 0.20

Inverts
Invert Upstream: 100.00 ft
Invert Downstream: 98.00 ft
Length: 200.00 ft
Slope: 0.010000 ft/ft

Headwater Elevations
Maximum Allowable: 110.00 ft
Computed Headwater: 109.43 ft
Inlet Control: 109.43 ft
Outlet Control: 108.16 ft

Exit Results
Discharge: 225.00 cfs
Velocity: 15.07 ft/s
Depth: 4.00 ft

OK   Cancel   Output   Solve   Help

Figure 3.2 Culvert Calculator Screen

Output:
Use a 54-inch diameter concrete pipe with allowable HW = 10.0 feet and
actual HW = 9.43 feet respectively for the 50 year storm flows, and a maximum outlet
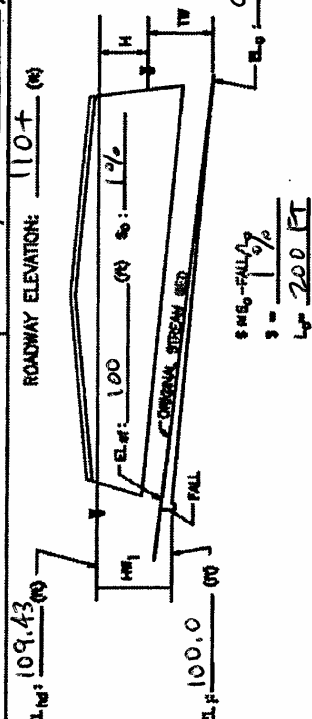velocity of 15.07 feet per second.

Figure 3.3 Culvert Design Worksheet for Example #1 (Federal Highway Administration, HDS, No.5)

3.2.4.2 Example 2 – Chart Procedure: From the American Concrete Pipe Association (www.concretepipe.com)

List Design Data:
A.   Q25 = 180 cubic feet per second
         Q50 = 225 cubic feet per second
B.   L = 200 feet
C.   So = 0.01 feet per foot
D.   Allowable HW = 10 feet for 25 and 50-year storms
E.   TW = 3.5 feet for 25-year storm
         TW = 4.0 feet for 50-year storm
F.   Circular concrete culvert with a projecting entrance, n = 0.012

Select Culvert Size:
A.   Try D = HW/2.0 = 10/2.0 = 5 feet or 60 inch diameter as first trial size.
B.   In Figure 3.5, project a vertical line from Q = 180 cubic feet per second
         to the inlet control curve and read horizontally HW = 6.2. Since HW =
         6.2 is considerably less than the allowable try a 54-inch diameter.
         In Figure 3.4, project a vertical line from Q = 180 cubic feet per second
         to the inlet control curve and read horizontally HW = 7.2 feet.
         In Figure 3.4, project a vertical line from Q = 225 cubic feet per second
         to the inlet control curve and read horizontally HW = 9.6 feet.
C.   In Figure 3.4, extend the vertical line from Q = 180 cubic feet per
         second to the L = 200 feet outlet control curve and read horizontally
         HW + SoL = 8.0 feet.
         In Figure 3.4, extend the vertical line from Q = 225 cubic feet per
         second to the L = 200 feet outlet control curve and read horizontally
         HW + SoL = 10.2 feet.
         SoL = 0.01 X 200 = 2.0 feet.
         Therefore   HW = 8.0 - 2.0 = 6.0 feet for 25-year storm
         HW = 10.2 - 2.0 = 8.2 feet for 50-year storm
D.   Since the calculated HW for inlet control exceeds the calculated HW
         for outlet control in both cases, inlet control governs for both the 25-year
         and 50-year storm flows.

Determine Outlet Velocity:
B.   Enter Figure 3.6 on the horizontal scale at a pipe slope of 0.01 feet per
         foot (1.0 feet per 100 feet). Project a vertical line to the line
         representing 54-inch pipe diameter. Read a full flow value of 210 cubic
         feet per second on the vertical scale and a full flow velocity of 13.5 feet
         per second. Calculate = Q50/QFull = 225/210 = 1.07.

Enter Figure 3.7 at 1.07 on the horizontal scale and project a vertical line to the "flow" curve. At this intersection project a horizontal line to the "velocity" curve. Directly beneath this intersection read $V_{50}/V_{full} = 1.12$ on the horizontal scale. Calculate $V_{50} = 1.12\ V_{Full} = 1.12 \times 13.5 = 15.1$ feet per second.

Record Selection:

Use a 54-inch diameter concrete pipe with allowable HW = 10.0 feet and actual HW = 7.2 and 9.6 feet respectively for the 25 and 50 year storm flows, and a maximum outlet velocity of 15.1 feet per second.

CULVERT CAPACITY
54-INCH DIAMETER PIPE
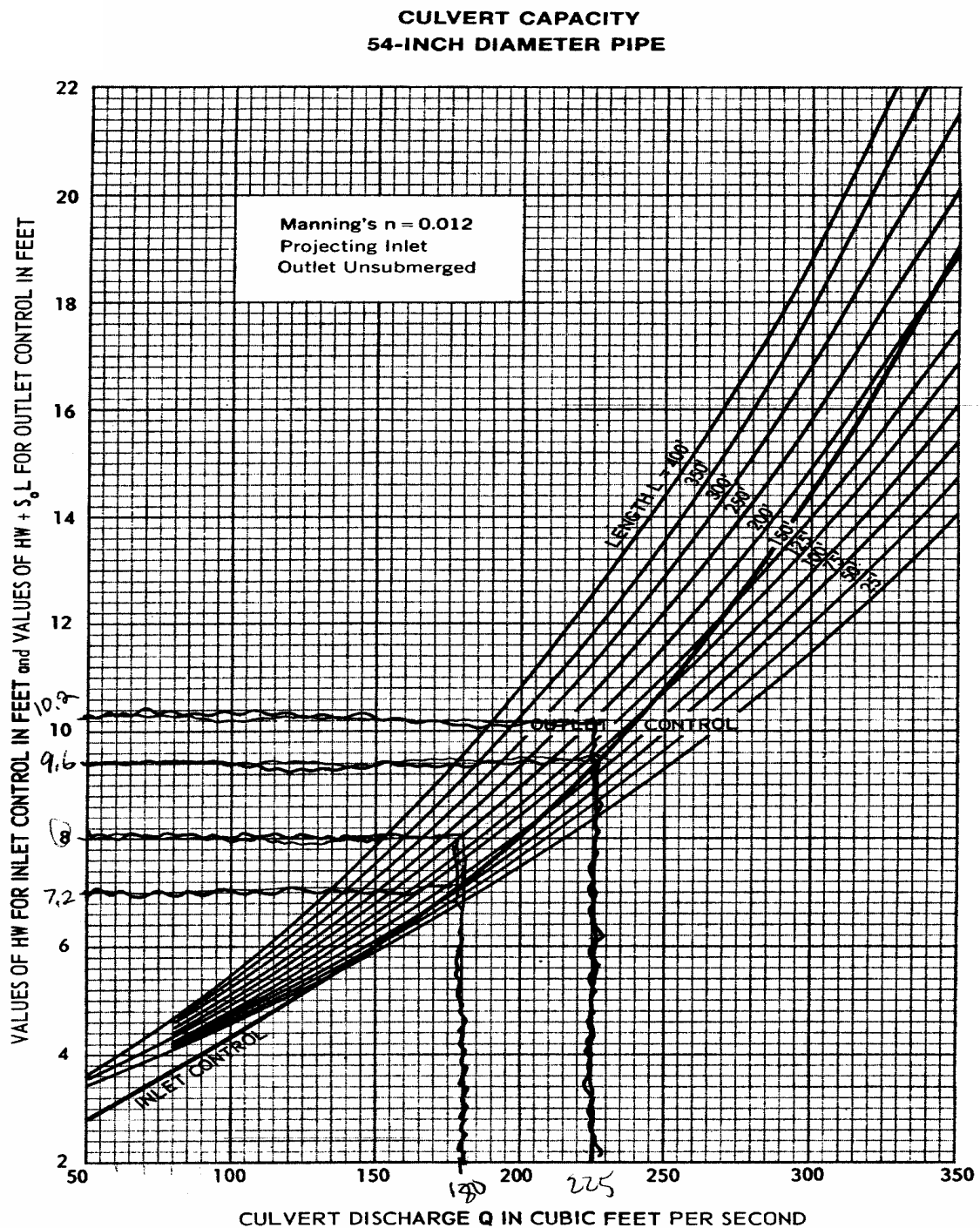
Figure 3.4 Culvert Capacity Design Chart - 54-inch (Concrete Pipe Design Manual)

CULVERT CAPACITY
60-INCH DIAMETER PIPE
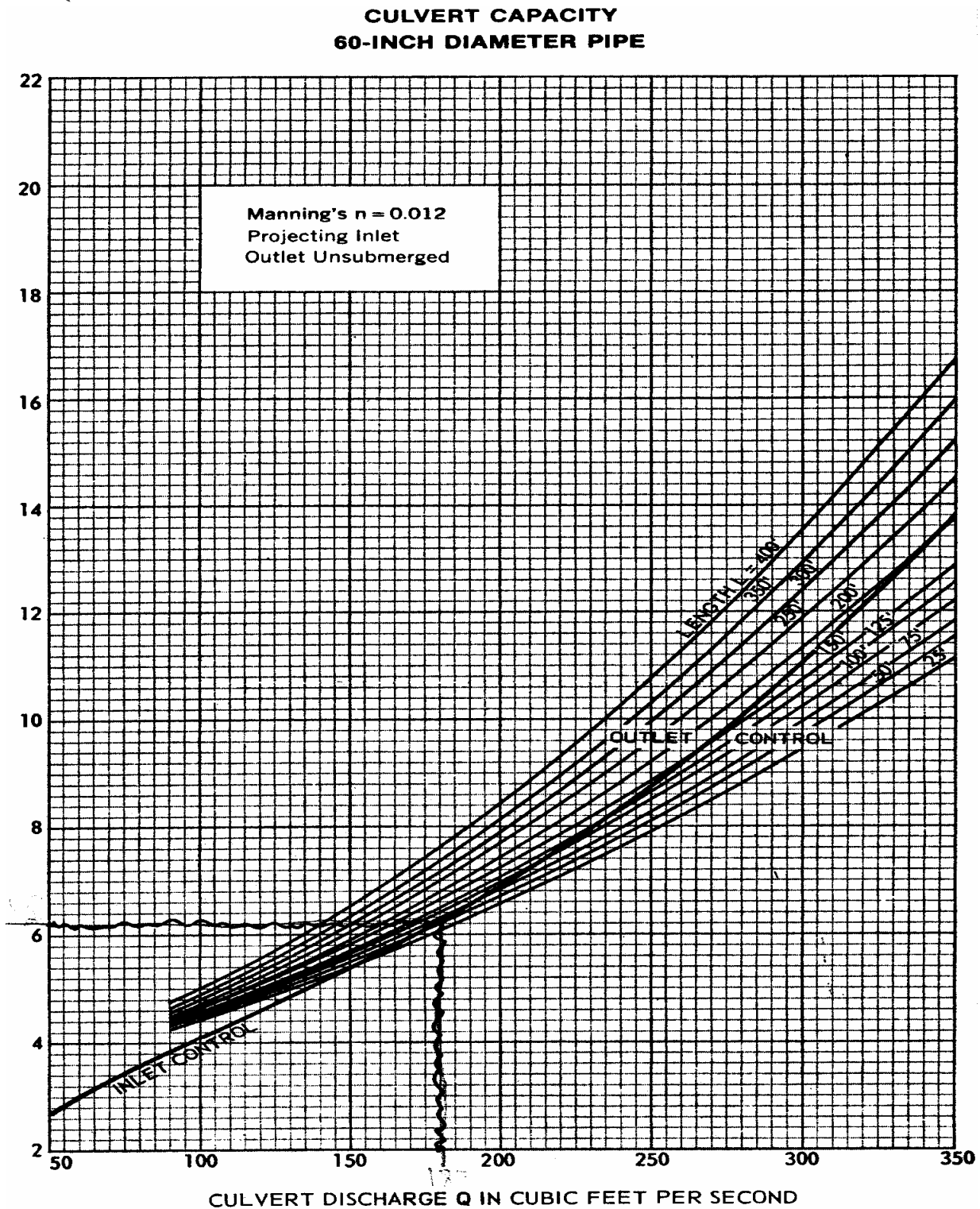
Figure 3.5 Culvert Capacity Design Chart – 60-inch (Concrete Pipe Design Manual)

FLOW FOR CIRCULAR PIPE FLOWING FULL
BASED ON MANNING'S EQUATION n=0.012
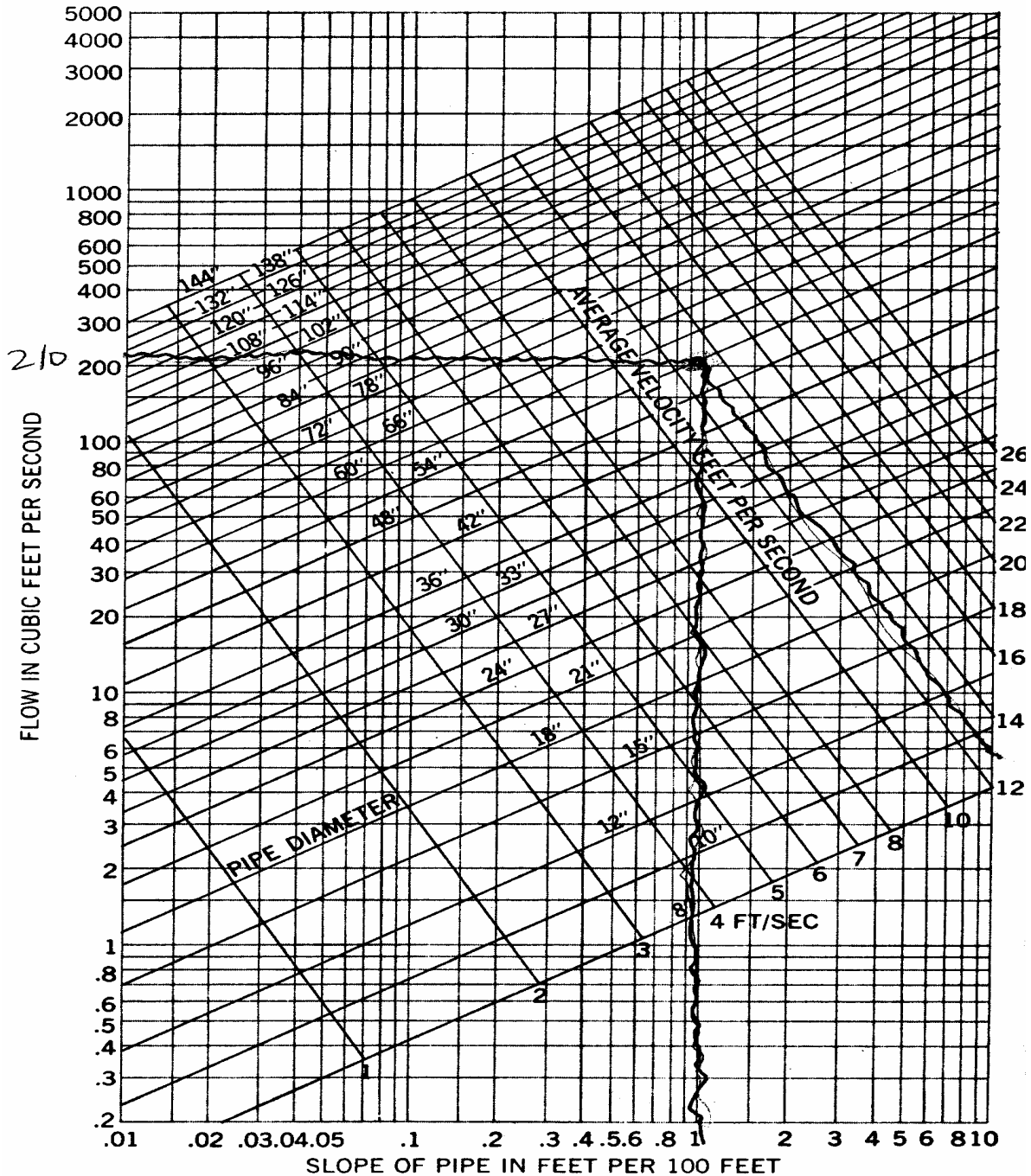
Figure 3.6 Flow for Circular Pipe Flowing Full (Concrete Pipe Design Manual)

RELATIVE VELOCITY AND FLOW IN
CIRCULAR PIPE FOR ANY DEPTH OF FLOW
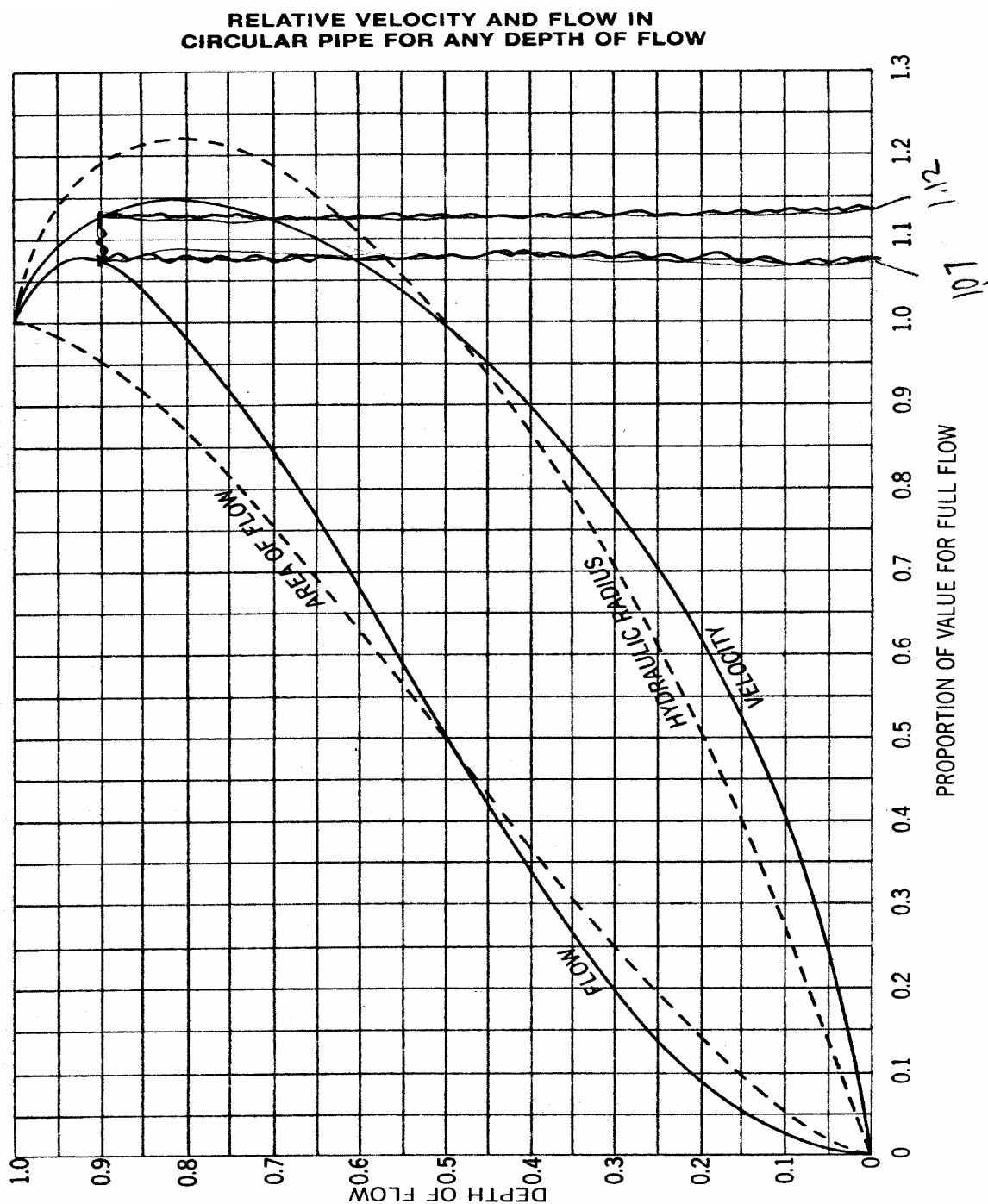
Figure 3.7 Circular Pipe for any Depth of Flow (Concrete Pipe Design Manual)

CULVERT DESIGN FORM

PROJECT: EXAMPLE #2 — CHART PROCEDURE

STATION: N/A

DESIGNER/DATE: GWB / 2/9/2

SHEET 1 OF 1

REVIEWER/DATE: ___ / ___

EL_hd = 109.6 (ft)

ROADWAY ELEVATION: 110.4 (ft)

EL_o = 98.0 (ft)

EL_f = 100.0 (ft)

(DESIGN)

S_o · L_o = FALL

S = 1%

L_o = 200 FT

HYDROLOGICAL DATA

☑ METHOD: (GIVEN)
☐ DRAINAGE AREA: ___   ☑ STREAM SLOPE: 1%
☐ CHANNEL SHAPE: ___
☐ ROUTING: ___   ☐ OTHER: ___

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
25	180	3.5
50	225	4.0

CULVERT DESCRIPTION:

MATERIAL – SHAPE – SIZE – ENTRANCE

HEADWATER CALCULATIONS

	TOTAL FLOW Q (cfs)	FLOW PER CHANNEL Q/N (cfs)	INLET CONTROL HW_i/D	INLET CONTROL HW_i	FALL (ft)	EL_hi (ft)	TW (ft)	d_c	$\frac{d_c+D}{2}$	h_o	k_e	H (ft)	EL_ho (ft)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY FPS	COMMENTS
54" SINGLE	180	180	1.6	7.2	2.0	100	4.0	3.9	4.2	4.2	0.2	3.8	106	IN	13.5	V_TW TOO HIGH
" (DESIGN)	225	225	2.14	9.6	2.0	100	4.0	4.2	4.35	4.35	0.2	5.9	108.25	IN	15.1	V_TW TOO HIGH

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW_i/D = HW_i/D OR HW_i/D FROM DESIGN CHARTS

(3) FALL = HW_i – (EL_hd – EL_sf); FALL IS ZERO FOR CULVERTS ON GRADE

(4) EL_hi = HW_i + EL_i (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or (d_c + D/2)(WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (29n^2 L)/R^{1.33}] V^2/2g$

(8) EL_ho = EL_o + H + h_o

COMMENTS / DISCUSSION:

VELOCITIES EXCEED 10 FPS, REDESIGN AND REANALYZE

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

CULVERT BARREL SELECTED:

SIZE: 54" – SINGLE

SHAPE: CIRCULAR

MATERIAL: RCP

ENTRANCE: PROJECTING

Figure 3.8 Culvert Design Worksheet for Example #2 (Federal Highway Administration, HDS, No.5)

3.2.4.3 Example 3- Nomograph Procedure: From the American Concrete Pipe
Association

List Design Data:
A.   Q25 = 180 cubic feet per second
       Q50 = 225 cubic feet per second
B.   L = 200 feet
C.   So = 0.01 feet per foot
D.   Allowable HW = 10 feet for 25 and 50-year storms
E.   TW = 3.5 feet for 25-year storm
       TW = 4.0 feet for 50-year storm
F.   Circular concrete culvert with a projecting entrance, n = 0.012
       Select Trial Culvert Size
       D = HW/2.0 = 10/2.0 = 5 feet

Determine Trial Culvert Headwater Depth:
A. Inlet Control:
       (1)   For Q = 180 cubic feet per second and D = 60 inches, Figure 3.9 indicates
               HW/D = 1.25. Therefore HW = 1.25 X 5 =6.2 feet.
       (2)   Since HW = 6.2 feet is considerably less than allowable try a 54-inch pipe.
                       For Q = 180 cubic feet per second and D = 54 inches,
                       Figure 3.9 indicates HW/D = 1.6.
                       Therefore HW = 1.6 X 4.5 = 7.2 feet.

                       For Q = 225 cubic feet per second and D = 54 inches,
                       Figure 3.9 indicates HW/D = 2.14.
                       Therefore HW 2.14 X 4.5 = 9.6 feet.
B. Outlet Control:
       (1) TW = 3.5 and 4.0 feet is less than D = 4.5 feet.
       (2) Table 3.4, ke, = 0.2.
                       For D = 54 inches, Q = 180 cubic feet per second,
                       Figure 3.11 indicates dc, 3.9 feet which is less than D = 4.5 feet.
                       Calculate ho = (dc + D)/2 = (3.9 + 4.5)/2 = 4.2 feet.

                       For D = 54 inches, Q = 180 cubic feet per second, ke = 0.2 and
                       L = 200 feet. Figure 3.10 indicates H = 3.8 feet.
                       Therefore HW = 3.8 + 4.2 - (0.01 X 200) = 6.0 feet.

                       For D = 54 inches, Q = 225 cubic feet per second,
                       Figure 3.11 indicates dc = 4.2 feet which is less than D = 4.5 feet.
                       Calculate ho = (dc + D)/2 = (4.2 + 4.5)/2 = 4.35 feet.

For D = 54 inches, Q = 225 cubic feet per second, ke = 0.2 and
L = 200 feet. Figure 3.10 indicates H = 5.9 feet.
Therefore HW = 5.9 + 4.3 - (0.01 X 200) = 8.2 feet.

C. Inlet control governs for both the 25 and 50-year design flows. A 48-inch culvert
would be sufficient for the 25-year storm flow but for the 50-year storm flow the HW
would be greater than the allowable.

Determine Outlet Velocity:
B.   Enter Figure 3.6 on the horizontal scale at a pipe slope of 0.01 feet per foot.
Project a vertical line to the line representing 54-inch pipe diameter.
Read a full flow value of 210 cubic feet per second on the vertical scale and a full
flow velocity of 13.5 feet per second.
Calculate Q50/QFull = 225/210 = 1.07.
Enter Figure 3.7 at 1.07 on the horizontal scale and project a vertical line to the
"flow" curve. At this intersection project a horizontal line to the "velocity"
curve. Directly beneath this intersection read V50/VFull = 1.12 on the
horizontal scale.
Calculate V50 = 1.12 X VFull = 1.12 X 13.5 = 15.1 feet per second.

Record Selection:
Use a 54-inch diameter concrete pipe with allowable HW = 10.0 feet and actual
HW = 7.2 and 9.6 feet respectively for the 25 and 50-year storm flows,
and a maximum outlet velocity of 15.1 feet per second.

Figure 3.9 Headwater Depth for Circular Concrete Pipes – Inlet Control
(Bureau of Public Roads)

Table 3.4 Entrance Loss Coefficients (Bureau of Public Roads)

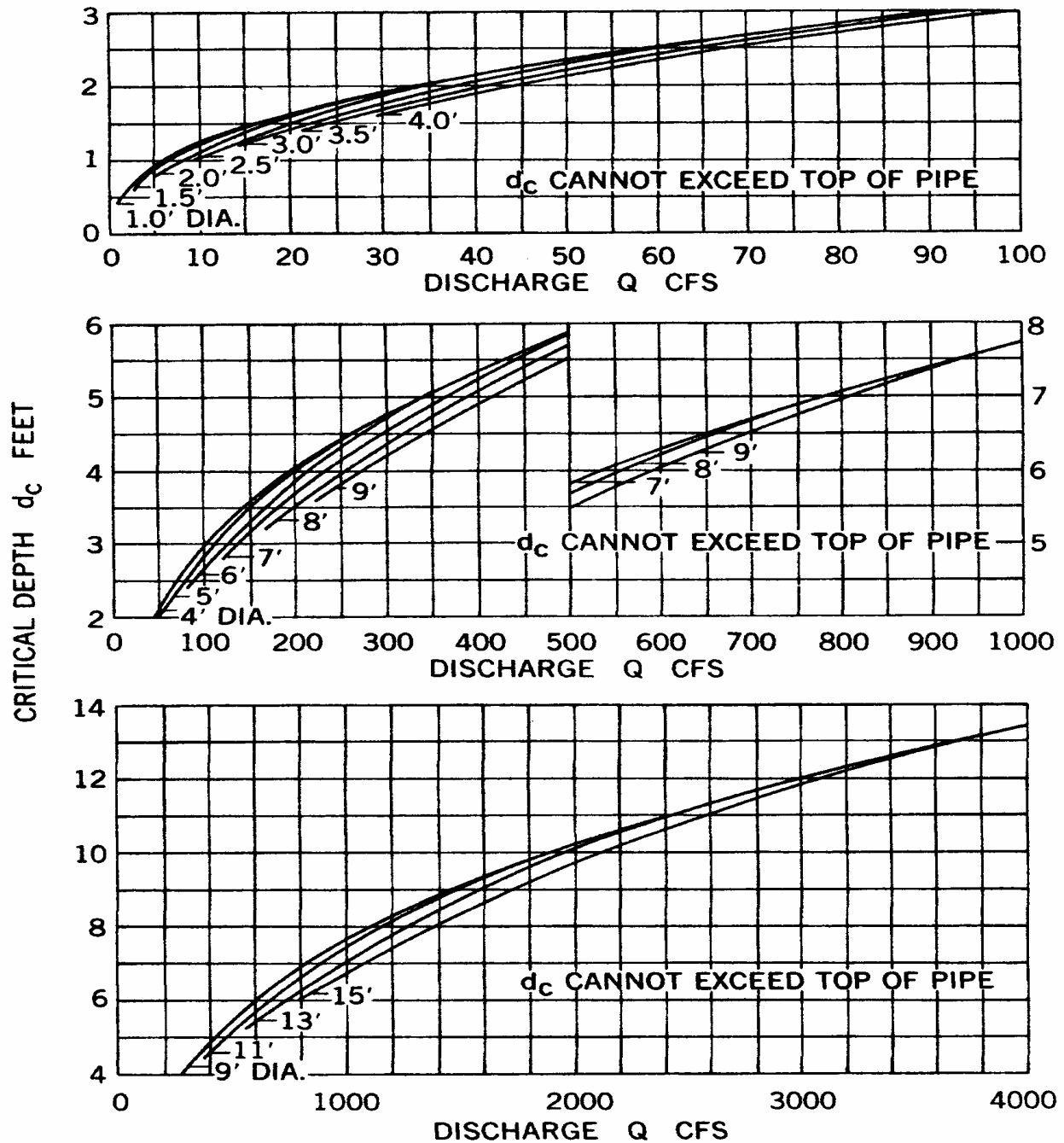## ENTRANCE LOSS COEFFICIENTS

Coefficient k_e to apply to velocity head $\dfrac{V^2}{2g}$ for determination of head loss at entrance to a structure, such as a culvert or conduit, operating full or partly full with *control at the outlet.*

Entrance head loss $H_e = k_e \dfrac{V^2}{2g}$

TYPE OF ENTRANCE	COEFFICIENT k_e
Projecting from fill, socket end (groove-end).	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end).	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
End-Section conforming to fill slope	0.5

Note: "End Section conforming to fill slope" are the sections commonly available from manu-facturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance.

TYPE OF STRUCTURE AND DESIGN OF ENTRANCE BOX, REINFORCED CONCRETE	COEFFICIENT k_e
Headwall parallel to embankment (no wing walls)	
Square-edged on 3 edges.	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wing walls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension	0.2
Wing walls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wing walls parallel (extension of sides)	
Square-edged at crown	0.7

Figure 3.10 Critical Depth for Circular Pipes
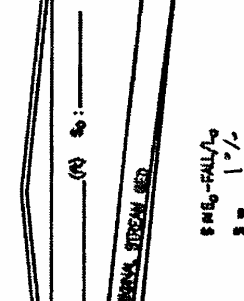
HEAD FOR CIRCULAR CONCRETE PIPE
CULVERTS FLOWING FULL
n = 0.012

SUBMERGED OUTLET CULVERT FLOWING FULL
$HW = H + h_O - S_O L$

For outlet crown not submerged, compute HW by
methods described in the design procedure
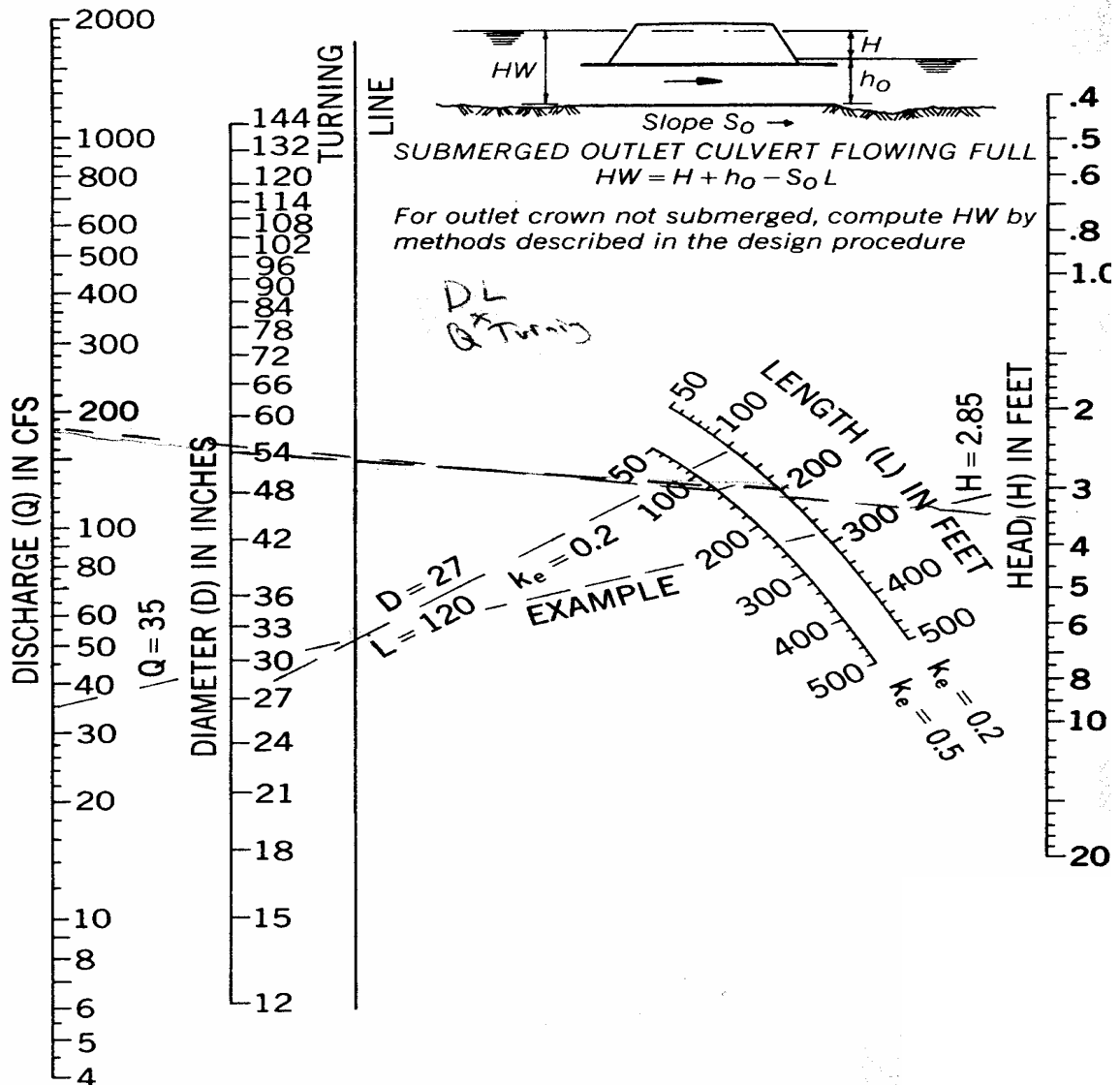
BUREAU OF PUBLIC ROADS   JAN. 1963

Figure 3.11 Circular Concrete Pipe Nomograph

CULVERT DESIGN FORM

PROJECT: EXAMPLE #3 — NOMOGRAPH PROCEDURE

STATION: N/A
SHEET 1 OF 1

DESIGNER/DATE: GWB / 2/9/3
REVIEWER/DATE: /

ROADWAY ELEVATION: 110.4 (ft)

EL$_{hd}$: 109.6 (ft)
EL$_i$: 100.0 (ft)

(DESIGN)

EL$_o$: 98.0 (ft)

S = 1%
L$_a$ = 200 ft

HYDROLOGICAL DATA

☑ METHOD: GIVEN
☐ DRAINAGE AREA: ☑ STREAM SLOPE: 1%
☐ CHANNEL SHAPE:
☐ ROUTING: ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW(cfs)	TW(ft)
25	180	3.5
50	225	4.0

CULVERT DESCRIPTION:
MATERIAL — SHAPE — SIZE — ENTRANCE

	TOTAL FLOW	FLOW PER BARREL	INLET CONTROL			OUTLET CONTROL						CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
YR	Q (cfs)	Q/N (cfs)	HW/D	HW	FALL (ft)	EL$_{hi}$ (ft)	TW (ft)	d$_c$	$\frac{d_c+D}{2}$	h$_o$	k$_e$	H (ft)	EL$_{ho}$ (ft)		

HEADWATER CALCULATIONS

54" SINGLE:
| 25 | 180 | 180 | 1.6 | 7.2 | 2.0 | 100 | 3.5 | 3.9 | 4.2 | 4.2 | 0.2 | 3.8 | 106 | IN | 13.5 | V TOO HIGH |

" (DESIGN) 50:
| 50 | 225 | 225 | 2.14 | 9.6 | 2.0 | 100 | 4.0 | 4.2 | 4.35 | 4.35 | 0.2 | 5.9 | 108.5 | IN | 15.1 | V TOO HIGH |

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW$_i$/D = HW/D OR HW$_i$/D FROM DESIGN CHARTS

(3) FALL = HW$_i$ − (EL$_{hd}$−EL$_{sf}$); FALL IS ZERO FOR CULVERTS ON GRADE

(4) EL$_{hi}$ = HW$_i$ +EL$_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h$_o$ = TW or (d$_c$ +D/2)(WHICHEVER IS GREATER)

(7) H = [1+k$_e$+(29n^{2}L)/R$^{1.33}$]v^2/2g

(8) EL$_{ho}$ = EL$_o$ + H + h$_o$

SUBSCRIPT DEFINITIONS:
a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

VELOCITIES EXCEED 10 FPS,
REDESIGN AND REANALYZE

CULVERT BARREL SELECTED:
SIZE: 54" - SINGLE
SHAPE: CIRCULAR
MATERIAL: RCP
ENTRANCE: PROJECTING

Figure 3.12 Culvert Design Worksheet for Example #3 (Federal Highway Administration, HDS, No.5)

3.2.5   EROSION CONTROL

Velocities through culverts and at their discharge point are of concern and must be properly managed to minimize potential erosion.

3.2.5.1   Culvert Velocities

Table 3.5 Culvert Velocities

Flow Through	Minimum Velocity (fps)	Maximum Through Velocity (fps)	Maximum Discharge Velocity (fps)
Culverts and Laterals	3	15	10*

* Greater than 5 fps requires additional erosion control as in 3.2.5.2.5 and 3.4.4.1.

3.2.5.2   Headwalls and Rip-Rap

Concrete headwalls or end-sections are required on all terminations of culverts or storm sewers in public ROW or public easements. Additional outlet structures are required when discharge velocities exceed 5 fps.

3.2.5.2.1 Selecting Headwalls and Rip-Rap

The following design parameters provide guidance in selecting the proper culvert end-treatment:

3.2.5.2.2 Parallel Headwall or Sloped End-treatment:

1. Approach or discharge velocities are less than 6 fps.
2. Backwater pools may be permitted.
3. Approach channel is undefined.
4. Sufficient ROW is available.
5. Downstream channel protection is not required.

3.2.5.2.3 Flared Headwall:

1. Approach or discharge velocities are between 6 and 10 fps.
2. Channel is well defined.
3. Medium amounts of debris exist.

3.2.5.2.4 *Warped Headwall:*

1. Approach or discharge velocities are between 8 and 10 fps.
2. Channel is well defined and concrete lined.
3. Medium to large amounts of debris exist.

3.2.5.2.5 Rip-Rap (Rock Rubble):

1. Approach and Discharge velocities between 5 and 8 fps.
2. Refer to Figure 3.13 Outlet Velocity Control Structure on the following page.
3. The riprap size and area coverage can be determined using the following chart procedure referring to the chart in Figure 3.14 Culvert Design Curves.

Example:
Given: Pipe Diameter = D_o = 24 in.
        Design Flow = 30 cfs

Solution:
        Size from Figure 3.14:
                        d_{50} = 0.3 ft. = 4 in.
                        $d_{max.}$ = (1.5) x (d_{50}) = 6 in.
                        Rip-rap size = (1.5) x ($d_{max.}$) = 9 in.

        Length from Figure 3.14:
                        L_a = 30 ft. minimum

        Widths: 3 D_o = 3 x 24 in. = 72 in. minimum upstream width
                                   D_o + 0.4 L_a = 24 in. + (0.4 x 30 ft.) = 14 ft.
                                           minimum downstream width

PLAN

HEADWALL SHOWN,
FOR OTHER HEADWALLS
OR END OF CONC. CHANNEL LINING.

TOP OF CHANNEL BANK

Do

T   0%

La

SECTION A-A

FILTER FABRIC (TYP.)

WELL GRADED
STONE RIP-RAP
(TYP.)

Do

SECTION B-B

RIP-RAP TO TOP OF
CHANNEL BANK OR
1' ABOVE DESIGN
FLOW DEPTH
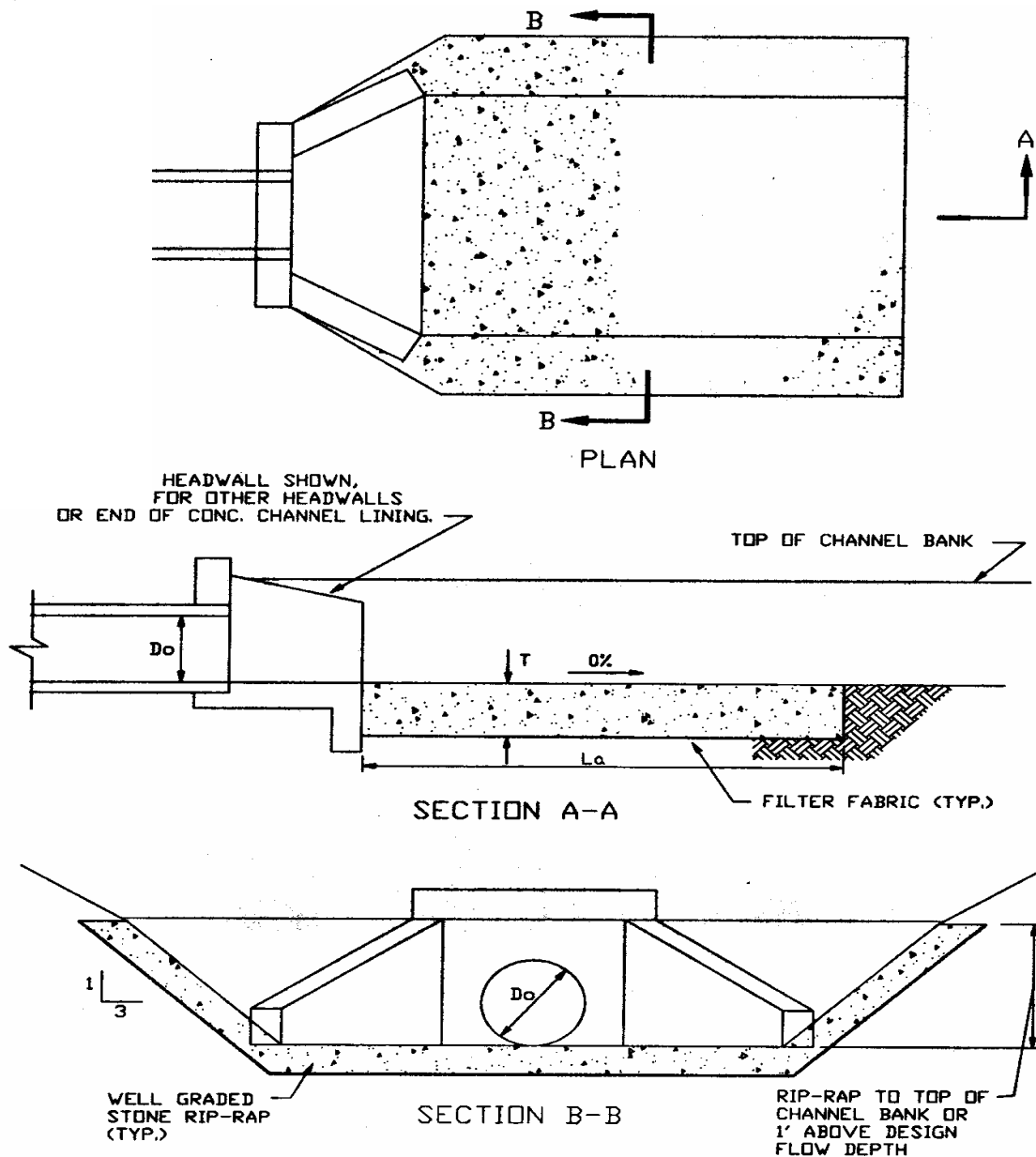
OUTLET VELOCITY CONTROL STRUCTURE
N.T.S.

Figure 3.13 Outlet Velocity Control Structure (NTS) (City of Plano, Texas)
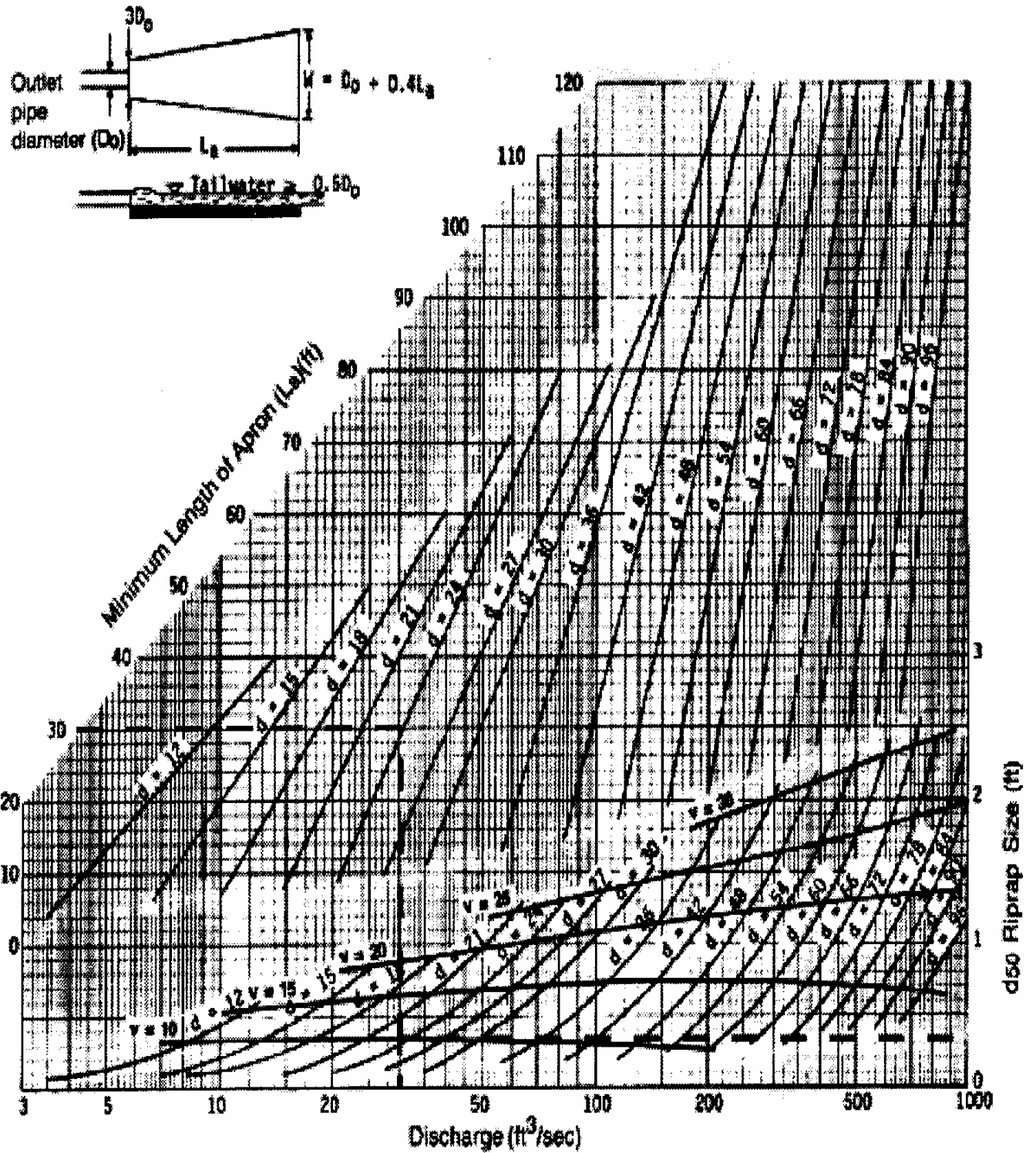
Figure 3.14 Culvert Design Curves (City of Plano, Texas)

Curves may not be extrapolated.

3.2.6 SUBMITTAL REQUIREMENTS:

1. Design Method with calculations, charts, and or nomographs
2. Digital copy if software is used.
3. Culvert Worksheet (Figure 3.15).
4. Site Plan or Construction Plans with Plan and Profile with surrounding grades all to scale.
5. Contour Map.
6. Vicinity Map.
7. FEMA note stating whether Floodplain exists on site and corresponding panel number.
8. Engineer's certification (refer to **INTRODUCTION**).

Figure 3.15 Culvert Design Worksheet (Blank) (Federal Highway Administration, HDS, No.5)

3.3   BRIDGES

3.3.1   BRIDGE LAYOUT GEOMETRY

3.3.1.1 Minimum Chord Elevation

The minimum chord elevation is defined as the bridge deck bottom elevation. This elevation shall be the highest of one foot above the 100-year flood elevation (a 1-foot freeboard) and the surrounding natural ground. However, in some cases to avoid an increase in downstream flows, the City Engineer may allow no freeboard and a backwater condition.

3.3.1.2 Horizontal Layout

The bridge should be aligned to accommodate the existing channel location as well as the ultimate channel section. The bridge should also be designed to convey the storm waters 90 degrees to the street section. The City Engineer needs to be contacted to evaluate a proposed crossing that is skewed.

3.3.1.3 Piers Locations

Bridge piers should be located to minimize disturbance to the storm waters by keeping the piers out of the low flow section. Align and space the piers to minimize possible debris collection by aligning the piers parallel to the channel flow.

3.3.1.4 Easements / R.O.W.

Additional public easements or R.O.W. may be required to maintain bridges. This may include a flared R.O.W., which includes the headwall or riprap sections and the necessary maintenance access.

3.3.2 BRIDGE DESIGN

3.3.2.1 HEC-RAS / HEC-2

Bridges will require the use of the U. S. Army Corps of Engineers software HEC – 2 or HEC – RAS that is a backwater method to design and analyze a proposed bridge section. If the subject drainage reach is FEMA Floodplain Zone AE, detailed study information should be available to supplement a study. The City Engineer will determine if the proposed bridge will require a FEMA submittal (refer to Section 3.5 FLOODPLAINS for procedures).

3.3.2.2 No-Rise

Proposed bridges shall be designed so that the HEC study confirms the 100-year water surface elevation is not increased upstream or downstream.

3.3.2.3 Flow

The minimum design flow for bridge structures shall be a 100 yr. storm as in Section 1. The City Engineer may require larger design flows. If the subject channel reach is delineated as Floodplain Zone AE the flow data may be obtained from the FEMA Flood Insurance Study (FIS). Otherwise, a HEC-1 or HEC-HMS analysis may be required to determine the flow.

3.3.2.4 Scour Analysis

The City Engineer may require a HEC-18 Scour Analysis to ensure the bridge section is adequately designed against erosion. This publication is <u>Evaluating Scour at Bridges</u>, Hydraulic Engineering Circular No. 18 from FHWA.

3.3   OPEN CHANNELS / ROADSIDE DRAINAGE

3.4.1 OPEN CHANNEL GEOMETRY

3.4.1.1 Longitudinal Slopes

The minimum longitudinal slope shall be 0.7 %. The maximum longitudinal slopes shall be dictated by maximum velocities and the associated erosion control.

3.4.1.2 Side Slopes

Maximum side slopes shall be 4:1. The City Engineer may allow absolute maximum side slopes of 3:1.

3.4.1.3 Bottom Widths

The open channel bottom width shall be a minimum of 4 feet wide and the roadside ditch bottom width shall be a minimum of one-foot wide to accommodate maintenance.

3.4.1.4 Freeboard

A minimum of a one-foot freeboard shall be required above the design water surface elevation.

3.4.1.5 Berms

The City Engineer may require a proposed berm to achieve a structural compaction effort of 98% Standard Proctor. All side slopes for berms shall be a maximum of 3:1 for privately maintained berms and a maximum of 4:1 for publicly maintained berms. Refer to Easement subsection for additional maintenance berm guidelines. Table 3.6 provides minimum top of berm widths for a specific berm height.

Table 3.6 Minimum Top of Berm Widths

Berm Height	Minimum Top of Berm Width (ft.)
<= 2 feet	3
>2 feet & <6 feet	5
>6 feet	10

3.4.1.6 Bends

Man-made channels should have maximum deflection angle bends of 45 degrees. Erosion control will be required if the center line curve radius is less than 3 times the top width of the design flow.

3.4.1.7 Confluences

Channel confluences should be at 45 degrees or less.

3.4.2   OPEN CHANNEL DESIGN

Manning's Equation is an acceptable method and is commonly used for many applications. If a backwater or tailwater condition exists other methods such as TxDOT's *WinStorm* and the US Corp's *HEC-RAS* software should be utilized. For a brief discussion of WinStorm, refer to Section 4 and *HEC Studies*, refer to **FLOODPLAINS SECTION 3.5**.

3.4.2.1 Manning's Equation

Manning's Equation, or derivations of the equation, can be used for many culvert and open channel designs. These equations are acceptable for designs not involving headwater or tailwater influences. The following equations are the fundamental Manning's Equations:

$$Q = vA = \left(\frac{1.49}{n}\right) A \ R^{2/3}\sqrt{S} \qquad (3.1)$$

$$v = \left(\frac{1.49}{n}\right) R^{2/3}\sqrt{S} \qquad (3.2)$$

where:

Q	=	flow (cfs)
v	=	velocity (ft/s)
n	=	Manning's roughness coefficient (dimensionless)
R	=	A/WP = hydraulic radius (ft)
A	=	cross-sectional area (ft^2)
WP	=	wetted perimeter (ft)
S	=	slope of energy grade line (ft/ft), *or*
S	=	slope of channel bed *for uniform flow* (ft/ft)

3.4.2.2 Manning's 'n' Values for Channels and Floodplains

Table 3.7 *Manning's 'n' Values for Channels and Floodplains*

Channel			Floodplain			
Condition	Roughness Coefficient Component[1]		Condition	Roughness Coefficient		
				Min.	Normal	Max.
Material Involved			Pasture			
Earth		0.020	Short grass	0.025	0.030	0.035
Rock cut		0.025	High grass	0.030	0.035	0.050
Fine gravel	n_1	0.024				
Coarse gravel		0.028	Cultivated Areas			
			No crops	0.020	0.030	0.040
Degree of irregularity			Mature row crops	0.025	0.035	0.045
Smooth		0.000	Mature field crops	0.030	0.040	0.050
Minor	n_2	0.005				
Moderate		0.010	Brush			
Severe		0.020	Scattered brush, heavy weeds	0.035	0.050	0.070
Relative effect of obstructions			Light brush and trees (winter)	0.035	0.050	0.060
Negligible		0.000	Light brush and Trees (summer)	0.040	0.060	0.080
Minor	n_3	0.010 - 0.015				
Appreciable		0.020 – 0.030	Medium to dense Brush (winter)	0.045	0.070	0.110
Severe		0.040 – 0.060	Medium to dense Brush (summer)	0.070	0.100	0.160
Vegetation						
Low		0.005 – 0.010	Trees			
Medium		0.010 – 0.025	Dense willows	0.110	0.150	0.200
High	n_4	0.025 – 0.050	Cleared land w/ tree stumps, no sprouts	0.030	0.040	0.050
Very High		0.050 – 0.100	Same as above, but with heavy growth on sprouts	0.050	0.060	0.080
Degree of meandering			Heavy stand of Timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
Minor		1.000				
Appreciable	k	1.150				
Severe		1.300	Same as above, but with flood stage reaching branches	0.100	0.120	0.160

Source: Adapted from Chow (1959), After Walesh (1989)
1 The composite Manning roughness coefficient for a channel reach = $k(n_1 + n_2 + n_3 + n_4)$.

3.4.3 OPEN CHANNEL EXAMPLE

Given: The grassed trapezoidal channel shown is laid at a slope of 0.007 and a Manning's 'n' value of 0.030. The depth of flow is 2.0 ft. Assume uniform flow.
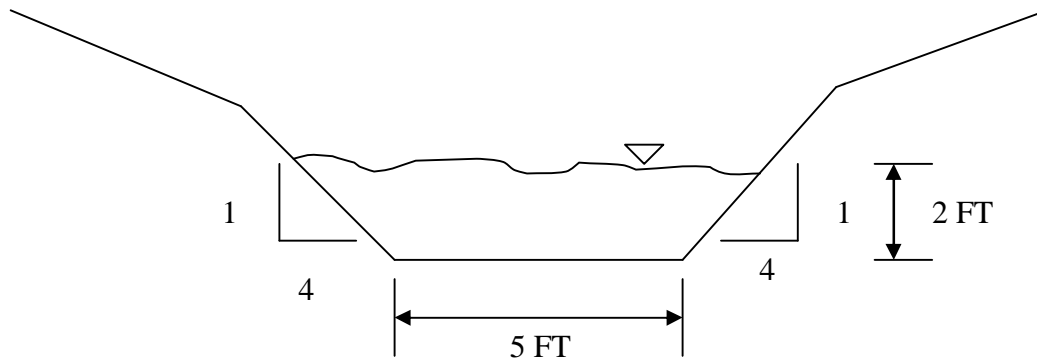
Find: Determine the flow rate and the velocity.

Solution: $\text{Area} = A = (5)(2) + \left(\dfrac{(2)(8)(2)}{2} \right) = 26\, sf$

$\text{Wetted Perimeter} = WP = 5 + 2\sqrt{2^2 + 8^2} = 21.5\, ft$

$\text{Hydraulic Radius} = R = \dfrac{A}{WP} = \dfrac{26}{21.5} = 1.2\, ft$

$\text{Flow} = Q = vA = \left(\dfrac{1.49}{n} \right) A\ R^{2/3} \sqrt{S} = \left(\dfrac{1.49}{0.030} \right)(26)(1.2)^{\frac{2}{3}}(0.007)^{0.5} = 122.1\, cfs$

$\text{Velocity} = v = \left(\dfrac{Q}{A} \right) = \left(\dfrac{122.1}{26} \right) = 4.7\, fps$

3.4.4   EROSION CONTROL

The use of rip-rap, low-flow channels, drop structures, baffles, stilling basins, back-slope swales, and geotextile mattings are some options that may be utilized to control erosion and may be required for a specific design. Refer to Section 3.2.5.2.5 for riprap design.

3.4.4.1 Open Channel Velocities

Table 3.8 Open Channel Velocities

Channel Lining	Design Flow Maximum Velocity (fps)
Grass	5
Rip-Rap (Rock Rubble)	8
Concrete	12

Velocities in excess of 12 fps shall require additional methods such as baffles, stilling basins, and drop structures to reduce to the velocities within Table 3.8.

3.4.5   EASEMENTS

Channels will only be accepted if improved in accordance with the Design Guidelines or accepted by the City Engineer. For maintenance, a 20-feet wide minimum Lateral Access Maintenance Easement shall be provided for every 300 feet of channel reach. Table 3.9 gives a minimum easement width for all open channels.

Table 3.9 Easement Widths for Open Channels

Channel Lining	Design Top of Bank Width (ft.)	Minimum Maintenance Berm Width	Minimum Easement Width (ft.)
Grass	15 or less	10 ft., one side	25
Grass	15 to 30	15 ft., one side	40 to 50
Grass	30 to 60	15 ft., each side	60 to 90
Grass	60 or greater	20 ft., each side	100
Concrete	All	10 ft. and 20 ft.	50

3.4.6 SUBMITTAL REQUIREMENTS:

1. Open Channel Method(s) and calculations.
2. Digital copy if software is used.
3. Site Plan or Construction Plans with Plan and Profile with surrounding grades all to scale.
4. Contour Map.
5. Vicinity Map.
6. FEMA note stating whether Floodplain exists on site and corresponding panel number.
7. Engineer's certification (refer to **INTRODUCTION**).

3.5   FLOODPLAINS

3.5.1   INTRODUCTION

The City of Bryan is a Participating Community within the National Flood Insurance Program (NFIP) to enable residents and business owners to qualify for nationally subsidized flood insurance. The Federal Emergency Management Agency (FEMA) has designated flood-prone areas within our community which are depicted on the Flood Insurance Rate Map (FIRM) with the associated detailed studies in the Flood Insurance Study (FIS). The flood-prone areas depicted are the 500-year Floodplain, the 100-year Floodplain, and the Floodway. The 100-year Floodplain and Floodway are regulated by the City in compliance with the FEMA's minimum regulations to allow the City of Bryan to be a Participating Community.   The minimum regulations have been adopted or exceeded with the City of Bryan's Storm Water Management Ordinance. Texas is within Region VI of FEMA with contact information below:

FEMA, Region VI
Federal Center
800 North Loop 288
Denton, TX 76201-3698
(940) 898-5136

NFIP, Region VI
Federal Sector – Civil Group
15835 Park Ten Place, Suite 108
Houston, TX 77804
(281) 829-6880

3.5.2   REGULATORY FLOODPLAIN

The 100-year Floodplain, also referred to as the **Special Flood Hazard Area** (**SFHA**)**,** is the extent of inundation resulting from a 100-year rainfall (**Base Flood**) which has a one percent probability of occurring in any given year. Unless otherwise specified, the term "Floodplain" refers to the 100-year Floodplain. The water surface associated with the Base Flood is known as **the Base Flood Elevation** (**BFE**).

3.5.3   REGULATORY FLOODWAY / FLOODWAY FRINGE

In general the floodway is the main stream channel which conveys most of the stream flow and also has the highest velocities. More specifically, the **Regulatory Floodway** is determined by encroaching the Floodplain model until a maximum of a one-foot surcharge above the BFE is produced.   The Floodway is the area inside the encroachments and the area outside the encroachments is known as **Floodway Fringe** (See Figure 3.16).
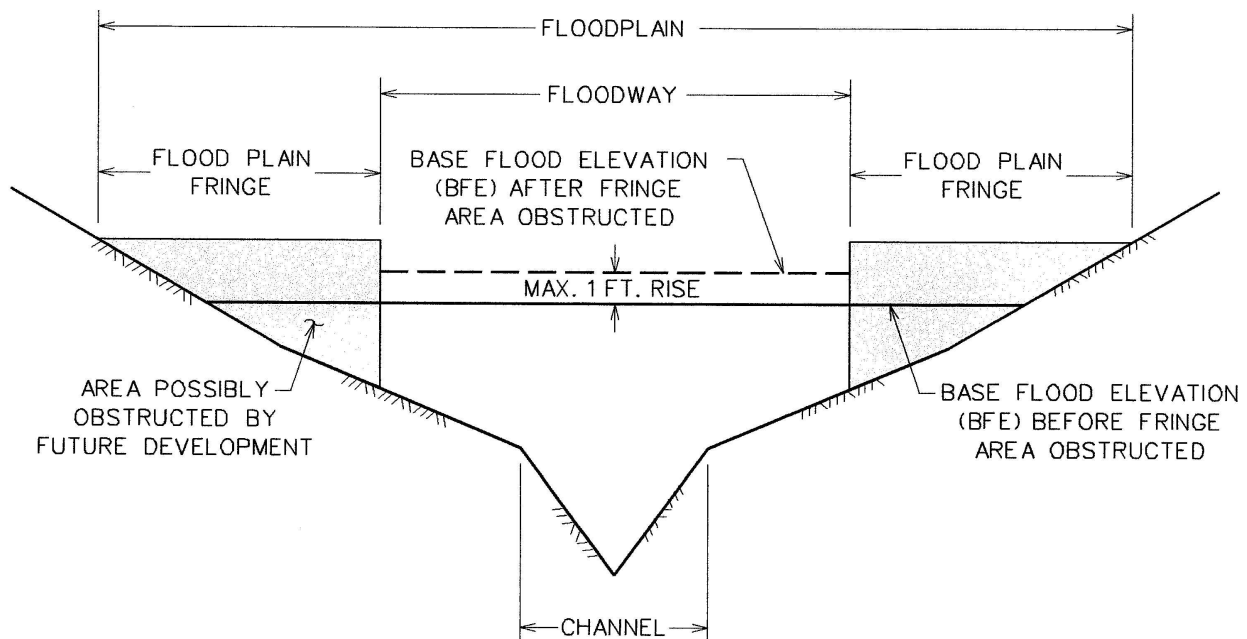
Figure 3.16 Floodplain Cross-section

3.5.4   DETERMINATION OF FLOODPLAIN / FLOODWAY

The FEMA FIRM should be the first reference to determine the boundary of the Floodplain or Floodway. For more accurate information, the FEMA FIS should be referenced next. Often a development site should have a topographic survey performed on which the BFE from the FIRM and FIS can be extended. Additional flood studies may be performed or referenced which provide "Better Data". Better Data are such studies which are recognized by the City of Bryan, however is not reflected on the FIRM or FIS. It is the City Engineer / Floodplain Administrator's authority to make the final determination or interpretation based on the existing data.

3.5.5   ENCROACHMENT

An encroachment within the Floodplain or Floodway is defined as any development or alteration including but not limited to a structure, a substantial improvement, fill, or excavation. (Additional regulations for encroachments are provided with the Stormwater Ordinance.)

3.5.5.1 Encroachment in the Floodway Fringe

A proposed encroachment within the Floodway Fringe may not create more than a one-foot surcharge above the BFE at any point along the stream. A detailed HEC-RAS, HEC-2, or other approved water modeling software will be required to justify the encroachment. If released for construction, the lowest finish floors of structures shall be elevated a minimum of one foot above the BFE. A CLOMA, LOMA, CLOMR or LOMR (described below) may be required. The City Engineer / Floodplain Administrator may waive the detailed study for minor encroachments. Equal compensatory storage may be required.

3.5.5.2 Encroachment in the Floodway

A proposed encroachment within the Floodway may not create a surcharge with respect to the BFE at any point along the stream. This is referred to as the *No-Rise* regulation. A detailed HEC-RAS, HEC-2, or other approved water modeling software will be required to justify the encroachment. If released for construction, the lowest finish floors of structures shall be elevated a minimum of one foot above the BFE. A CLOMR or LOMR (described below) may be required. Equal compensatory storage will be required.

3.5.6   LETTERS OF MAP REVISION

3.5.6.1 CLOMA / LOMA

The *Conditional Letter of Map Amendment (CLOMA)* is issued by FEMA to amend the FIRM if there is a relatively minor amendment, such as topographic, where a "proposed"

structure is on portion of land is inadvertently included in a SFHA. The *Letter of Map Amendment (LOMA)* is similarly issued for lands or "existing" structures. Neither a CLOMA nor LOMA will be issued in Floodways or in areas that have been filled since the first NFIP map designating the SFHA.

3.5.6.1.1 Submittal Requirements:

1. A copy of the recorded Deed
2. A copy of the recorded Plat
3. A sealed Topographic Map with lowest adjacent finished grade
4. The BFE by FIS or other approved
5. Complete the form entitled "Request for Letter of Map Amendment"
6. (If multiple lots or structures) A current FIRM sealed with depicted parcel boundary.
7. Additional data such as structural reports or hydraulic calculations may be necessary.
8. Culvert Worksheet (Figure 3.15).
9. Digital copy if software is used.
10. Site Plan or Construction Plans with Plan and Profile with surrounding grades all to scale.
11. Contour Map.
12. Vicinity Map.
13. FEMA note stating whether Floodplain exists on site and corresponding panel number.
14. Engineer's certification (refer to **INTRODUCTION**).

The entire application will be reviewed by the City and upon approval the City will submit to FEMA for review. NFIP/FEMA, Region VI has currently contracted their review to PBS&J:

FEMA LOMA Depot
Attn: LOMA Manager
12101 Indian Creek Court
Beltsville, MD 20705
(800) 697-7275

3.6.5.2 CLOMR / LOMR / CLOMR-FILL / LOMR-FILL

The *Conditional Letter of Map Revision (CLOMR)* is issued by FEMA to give approval for "proposed" alterations of the stream or "proposed" structures. The *Letter of Map Revision (LOMR)* is issued to amend the FIRM for the stream alteration or the structure. Revisions only involving fill have the respective designation, LOMR-FILL, which involves a shorter review and lower fee.

3.5.6.2.1 Submittal Requirements (for FEMA):

1.   A copy of the recorded Deed
2.   A copy of the recorded Plat
3.   A sealed Topographic Map with lowest adjacent finished grade
4.   The BFE by FIS or other approved
5.   Complete the form entitled "Request for Letter of Map Amendment"
      Complete the form entitled "Community Acknowledgement of Request for Letter of Map Revision"
6.   (If multiple lots or structures) A current FIRM sealed with depicted parcel boundary.
7.   (If multiple lots or structures) A certification of the compaction, slope, erosion, and location of fill outside the Floodway.
8.   Additional data such as structural reports or hydraulic calculations may be necessary.
9.   Culvert Worksheet (Figure 3.15).
10.   Digital copy if software is used.
11.   Site Plan or Construction Plans with Plan and Profile with surrounding grades all to scale.
12.   Contour Map.
13.   Vicinity Map.
14.   FEMA note stating whether Floodplain exists on site and corresponding panel number.
15.   Engineer's certification (refer to Section 1.4 and 1.6).

The entire application will be reviewed by the City and upon approval the City will submit to FEMA for review to address provided above in Section 3.5.6.1.1.

3.5.7 DESIGN IN FLOODPLAIN

In general, also refer to the Flood Prevention Ordinance Ch. 10 of the City of Bryan Code of Ordinances.

3.5.7.1 HEC Studies

HEC-RAS and HEC-2 are the standard accepted methods accepted by FEMA for establishing Floodplains or Letters of Map Revision. The HEC software programs were developed by the U.S. Corps of Engineers for a variety Hydrologic and Hydraulic analyses. HEC-RAS and HEC-2 were developed to specifically analyze backwater effects for rivers and bridges. HEC-1 and HEC-HMS were developed to hydrologic analysis and design.

3.5.7.2 HEC Selected Input Parameters

1. TP40 and Hydro 35 Precipitation Data as provided in Section 2.4.1 (Table 2.5).
2. 10, 25, 50, 100, and 500 year rainfall runs.
3. Lag Times for the unit hydrograph should be computed using the SCS lag equation.
4. Rational Formula should be used for the peak Q drainage basins less than 50 acres.
5. Balanced triangular hydrograph for PH record in HEC-1 should be used for drainage basins between 50 and 200 acres and lag times less than 30 minutes.
6. SCS Type III, 24 hour duration storm should be used for drainage basins larger than 200 acres or lag times greater than 30 minutes.
7. Modified-Puls for Channel Routings and Puls may be used for steep slopes.
8. Losses should be computed using the SCS curve number method.
9. The SCS unit hydrograph technique is encouraged where no data is available to estimate other parameters.

3.6   U.S. WATERS

This subsection is intended to provide a very brief summary of the permitting processes of the U.S. Army Corps of Engineers and the Environmental Protection Agency (EPA) involving U.S. Waters.

3.6.1 CHANNEL IMPROVEMENTS: SECTION 10

The U.S. Army Corps of Engineers is directed by Congress under Section 10 of the Rivers and Harbors of 1899 (33 USC 403) to regulate all work or structures in or affecting the course, condition or capacity of navigable waters of the United States. The intent of this law is to protect the navigable capacity of waters important to interstate commerce.

Navigable waters of the United States are those waters that are subject to the ebb and flow of the tide and/or are presently used, or have been used in the past, or may be susceptible for use to transport interstate or foreign commerce. A determination of navigability, once made, applies laterally over the entire surface of the water body, and is not extinguished by later actions or events, which impede or destroy navigable capacity. This definition does not apply to authorities under the Clean Water Act which definitions are described under 33 CFR Parts 323 and 328.

In common language, a "Corps Permit" is required for significant channel improvements involving major channel realignment or major slope protection.

3.6.2   WETLANDS: SECTION 404

The U.S. Army Corps of Engineers is direct by Congress under Section 404 of the Clean Water Act (33 USC 1344) to regulate the discharge of dredged and fill material into all waters of the United States, including wetlands. The intent of the law is to protect the nation's waters from the indiscriminate discharge of material capable of causing pollution and to restore and maintain their chemical, physical and biological integrity.

Wetlands are areas where the frequent and prolonged presence of water at or near the soil surface drives the natural system meaning the kind of soils that form, the plants that grow, and the fish and/or wildlife communities that use the habitat. Swamps, marshes, and bogs are well-recognized types of wetlands. However, many important specific wetland types have drier or more variable water systems than those familiar to the general public. Some examples of these are vernal pools (pools that form in the spring rains but are dry at other times of the year), playas (areas at the bottom of undrained desert basins that are sometimes covered with water), and prairie potholes. Section 404 jurisdiction is defined as encompassing Section 10 waters plus their tributaries and adjacent wetlands and isolated waters where the use, degradation or destruction of such waters could affect interstate or foreign commerce.

In common language, wetlands are identified by a biologist or other qualified personnel locating the presence of certain plants or wildlife in marshy areas. The City may require an Environmental Assessment Report to state if wetlands are present on a given project. If wetlands are present and are proposed to be affected, a 404 permit will be required.

3.6.3   SUBMITTAL REQUIREMENTS

A permit review process controls regulated activities. An Individual Permit is usually required for potentially significant impact. However, for most discharges that will have only minimal adverse effects, the Army Corps of Engineers often grants up-front General Permits. These may be issued on a Nationwide, Regional, or State basis for particular categories of activities (for example, minor road crossings, utility line backfill, and bedding) as a means to expedite the permitting process.

For further information for development and permitting concerning U.S. Waters in the Bryan area, contact our U.S. Army Corps of Engineers District Headquarters, Forth Worth District as below:

<div align="center">

U.S. Army Corps of Engineers
Regulatory Branch, CESWF-EV-R
P.O. Box 17300
Fort Worth, TX 76102-0300
(817) 978-2681

</div>

Section 4

Secondary Drainage

4.1 INTRODUCTION

This section describes guidelines, which can be used to design roadside ditches, storm sewers, streets, inlets, and overland flow draining small drainage areas. The following subsections discuss standards and reoccurrence intervals for storms to be used to design different structures. The methods described in this section are preferred methods, which should be utilized in design.

4.2 ROADSIDE DITCHES

The following section provides guidelines for the design of roadside ditches within the City of Bryan.

4.2.1 GENERAL REQUIREMENTS

Roadside ditches shall be designed to convey the 25-year storm runoff rate. The design flow rates shall be calculated using the Rational Method described in *Section 2.2*.

4.2.2 DESIGN CONSIDERATIONS FOR ROADSIDE DITCHES

Roadside ditches shall be designed to the following requirements:

1. Minimum and Maximum velocities in the ditch shall be limited to Table 3.8, *Section 3.4.4.1*. If velocities are not maintained below acceptable levels erosion control as described in *Section 3.4.4* shall be used.
2. Ditches must completely contain flow with a 25-year water surface 6 inches below the top of ditch.
3. Top of bank must not be closer than 2 feet laterally to the shoulder of the roadway (edge of base course).
4. Culverts shall be designed and sized in accordance with the procedure outlined in *Section 3.2*. Minimum size is set at 18 inches and must be constructed of reinforced concrete pipe and have sloped safety end treatments if parallel to roadway.
5. Side slopes shall be no steeper than 4 horizontal to 1 vertical (with a desirable of 6:1).
6. The minimum longitudinal grade shall be 0.70 %
7. The maximum design depth shall be limited to 3 feet with a minimum depth of 1.5 feet. Where depths greater than 3 feet are required, the design will be governed by the guidelines set forth in *Section 3.4, Open Channels*.
8. Once constructed, all ditches must be completely vegetated prior to acceptance by the City of Bryan. This may require sodding of the ditch.
9. All computations and design drawings shall demonstrate that the ditch meets the above design standards.

4.3   STREETS

Design criteria for the collection and transport of rainfall runoff from public streets is based on a design frequency as shown in Table 1.1. During the design, the maximum allowable ponding width of water in the street is governed by the classification of the street. Table 4.1 shows the relationship between the street classification, allowable ponding width, and design storm frequency.

Table 4.1 Maximum Ponding Width for Streets

Street Classification	Design Storm	Maximum Ponding Width
Residential	10-year	Flow of water in gutters shall be limited so that one standard lane (12-feet) remains clear
Collector	25-year	Flow of water in gutters shall be limited so that two standard lanes (12-feet) remain clear (at least one 12-foot lane in each direction)
Arterial	50-year	Flow of water in gutters shall be limited so that two standard lanes (12-feet) remain clear (at least one 12-foot lane in each direction)

4.3.1   GUTTER FLOW

The "Street Section" in the City of Bryan Design Guideline Manual shall govern the transverse slope of the street section for streets. The maximum longitudinal slope of the street shall be such that the maximum velocity in the gutter does not exceed 8 feet per second.

In general, collectors and arterial streets require the installation of storm drain systems. A residential street shall have cross-valley gutters only at infrequent locations in accordance with good engineering practices.

Once the cross-section of the street has been determined, the capacity of the gutter can be determined. When using a normal crowned street section, the gutter cross section takes the form of a triangle with the curb forming a vertical leg of the section. The flow through this cross-section is classified as open channel flow and the design calculations are based on Manning's Equation (3.1) Since the hydraulic radius is not suitable for describing the street cross-sections, a modification to the hydraulic radius is required.

The equation in terms of cross-slope, longitudinal slope, roughness, and depth of flow is given by Equation 4.1.

$$Q = 0.56 \left[\frac{Z}{n} \right] S^{0.5} d^{2.67} \tag{4.1}$$

Where:

Q = discharge, cfs
Z = reciprocal of cross slope, $1/S_x$, feet per foot
n = Manning's roughness coefficient
S = longitudinal slope, feet per foot
d = depth of flow at curb or deepest point, feet

The width of the flow or spread in the triangular channel can be calculated by multiplying the depth of the curb at the deepest point, d, by the reciprocal of cross slope, Z.

4.3.2 GUTTER CAPACITY EXAMPLE

Find: The carrying capacity of a residential street.

Given: 6-inch vertical curb
              27 ft wide, back of curb to back of curb, residential street
              Straight crown section, 3% cross slope
              Longitudinal slope – 0.75%
              Roughness, n = 0.014

Solution:
1. Determine allowable pavement width inundation
   Table 4.1, one standard lane, 12-feet, must remain clear.

2. Calculate width of flow, and gutter flow depth.
   Allowable width of water = (26 ft– 12 ft)/2 = 7 feet
   Gutter flow depth, d = (7 ft x 0.03 ft/ft) = 0.21 ft = 2.52 in.

3. Calculate capacity of gutter

   Using equation 4.1, $Q = 0.56 \left[\frac{Z}{n} \right] S^{0.5} d^{2.67}$

   Z = $1/S_x$ = 1/.03 = 33.33

   Q = 0.56 x (33.33/0.014) x $(.0075)^{0.5}$ $(0.210)^{2.67}$

   Q = 1.79 cfs in each gutter

4.4   RETAINING WALLS

Retaining walls shall be designed to meet or exceed TxDOT design standards.

4.5   INLETS

Several types of inlets are recommended for use in the City of Bryan: Curb Inlets on Grade, Grate Inlets, and Combination Inlets. Where space is limited or to drain small areas in parking lots, grate inlets shall be used instead of slotted drains. All inlets shall be designed to carry at least the design storm frequency runoff depending on the street classification and design storm frequency shown in Table 4.1. The minimum length of curb inlets on grade, recessed curb inlets, and combination inlets shall be five feet (5')(Mont. County)

Curb inlets shall be spaced to handle the design storm discharge so that the hydraulic gradient in the storm sewer is six inches below the flow line gutter elevation. Inlets shall be spaced so that the maximum travel distance of water in the gutter does not exceed five hundred feet (500') in one direction for residential streets and three hundred feet (300') in one direction on major thoroughfares and streets within commercial developments. Curb inlets shall also be placed on side streets, which intersect major thoroughfares in all original designs or developments to intercept gutter flow prior to entering the thoroughfare.   Under special conditions and circumstances curb inlets may be required in other locations. (Mont. County)

All inlets should have a minimum clogging reduction factor of 25% of the discharge rate.

4.4.1   INLETS ON GRADE

4.4.1.1 Curb Inlets on Grade

Curb opening inlets on grade are effective in the drainage of roadway pavements and are typically free of clogging. Figure 4.1 shows the curb inlet with terms referenced in this section. The terms are defined in Section 4.4.2. All of the equations given in this section apply to a street, which has a uniform cross slope from the crown to the face of the curb.
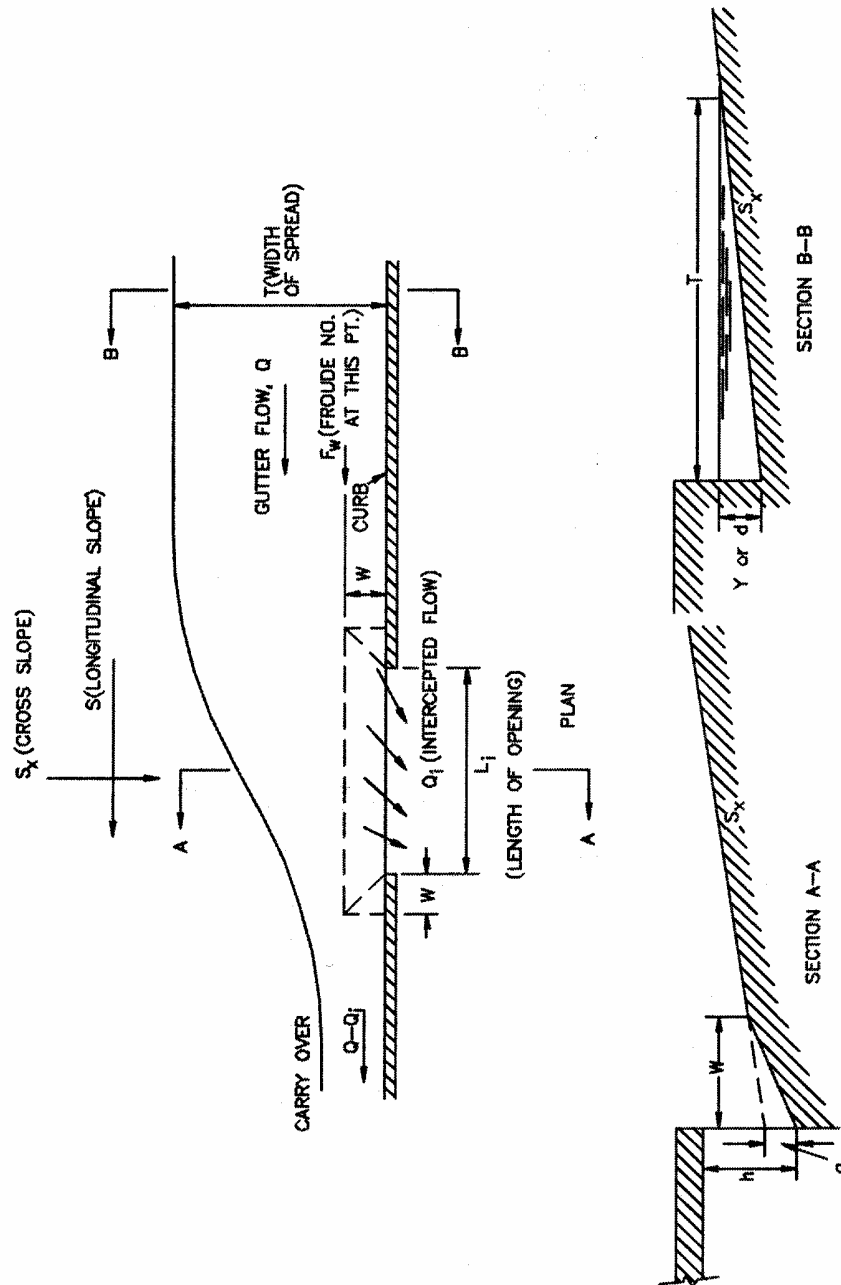
Figure 4.1 Inlet on Grade Factors

4.4.1.2 Calculating the Capacity of Curb-Opening Inlets on Grade

The ratio of the frontal flow to total gutter flow, E_o, for a straight cross slope is expressed by Equation 4.2.

$$E_o = \frac{Q_w}{Q} = 1 - (1 - W/T)^{2.67} \tag{4.2}$$

where:

E_o = Ratio of frontal flow to total gutter flow,
Q_w = Flow in width W, cfs
Q = Total gutter flow, cfs
W = Width of depressed gutter, feet
T = Total spread of water in gutter, feet

The length of the curb inlet required for total interception can be calculated using Equation 4.3.

$$L_r = K_c \, Q^{0.42} \, S^{0.3} \left(\frac{1}{n \, S_X} \right)^{0.6} \tag{4.3}$$

where:

K_c = 0.6 for English
L_r = length of curb inlet required, feet
Q = Total gutter flow, cfs
S = longitudinal slope, (ft/ft)
n = Manning's roughness coefficient
S_x = cross slope, (ft/ft)

For curb inlets shorter than the required length calculated in Equation 4.3, the efficiency of the inlet is calculated by Equation 4.4.

$$E = 1 - \left[1 - \frac{L_i}{L_T} \right]^{1.8} \tag{4.4}$$

where:

E = Efficiency of inlet or percentage of interception
L_i = Curb-opening length, ft
L_T = Curb-opening length required for 100% interception, ft

If the inlet being designed is in a depressed gutter section, then the length of the inlet required for total interception of the runoff can be determined by using Equation 4.5 and substituting S_e for S_x in Equation 4.3.

$$S_e = S_x + \frac{a}{W} E_o \qquad\qquad (4.5)$$

where:

S_e = equivalent cross slope, (ft/ft)
S_x = cross slope of the road, (ft/ft)
a = gutter depression depth, ft
W = gutter depression width, ft
E_o = ratio of depression flow to total flow, (Equation 4.2)

4.4.1.3 Curb Inlet Capacity Example

Find: A. The required length for 100% interception of runoff – No Depressed gutter
       B. The required length for 100% interception of runoff – Depressed gutter

Given: Flow in gutter – 1.47 cfs
           Pavement cross slope, S_x – 0.03 ft/ft
           27 ft wide, back of curb to back of curb, residential street
           Longitudinal slope – 0.75%
           Roughness, n = 0.014
           Gutter depression, a = 2 inches
           Gutter width, 2 feet
           Water flow width, 7 feet

Solution:

   A. $L_r = 0.6 \, (Q^{0.42}) \, (S^{0.30}) \, (1/(n(S_e)))^{0.6}$

         $L_r = 0.6 \, (1.47)^{0.42} \, (0.0075)^{.30} \, (1/(.014*.03))^{0.60}$

         L_r = 17.3 feet

B. $E_o = 1 - (1 - W/T)^{2.67}$

$E_o = 1 - (1 - (2/7))^{2.67}$

$E_o = 0.5928$

$S_e = S_x + (a/W)\, E_o$

$S_e = 0.03 + (2/12/2) * 0.5928$

$S_e = 0.0794$ ft/ft

$L_r = 0.6\, (Q^{0.42})\, (S^{0.30})\, (1/(n(S_e)))^{0.60}$

$L_r = 0.6\, (1.47^{.42})\, (.0075^{.30})\, (1/(0.014*.0794))^{0.60}$

$L_r = 9.6$ feet

4.4.1.4 Grate Inlets on Grade

The capacity of a grate inlet on grade depends on its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The design of a grate inlet on grade involves an analysis of given grate dimensions to estimate the interception rate. The ratio between the intercepted flow and the total flow in the gutter is the efficiency of the grate inlet. The following procedure is used to calculate the capacity of a grate inlet.

First the ratio of discharge over the grate inlet (frontal flow) to total flow is calculated using Equation 4.2. Next the ratio of the side flow, Q_s, to total gutter flow is given by Equation 4.6.

$$Q_s = Q\,(1 - E_o) \qquad\qquad (4.6)$$

where:

Q_s = ratio of side flow, cfs
Q = total gutter flow, cfs
E_o = ratio of frontal flow to total gutter flow, Equation 4.2

The ratio of frontal flow intercepted to total frontal flow is given by Equation 4.7.

$$R_f = 1 - 0.30\,(V - V_o)\,,\text{if } V > V_o$$
$$R_f = 1\,,\text{if } V < V_o \tag{4.7}$$

where:

R_f = ratio of frontal flow intercepted to total frontal flow
V = approach velocity in gutter, ft/s
V_o = minimum velocity that will cause splash over grate, ft/s

The splash over velocity is determined by Equation 4.8 in combination with Table 4.2.

$$V_o = A\,L^N\,factor^{1-N} \tag{4.8}$$

where:

V_o = splash over velocity, ft/s
A = constant for different grate types, Table 4.2
N = power coefficient for different grate types, Table 4.2
L = grate length, ft
Factor = 1 for English units

Table 4.2 Factors for Grate Capacity Calculations

Grate Type	Coefficient A	Power Coefficient N
Parallel	5.74872	0.5038679
Narrow	4.54822	0.5058875
Curved Vane	3.92812	0.5954068
45 Tilt Bar	3.35159	0.5926234
30 Tilt Bar	2.67291	0.7567052
Transverse	3.01181	0.6454720
Reticuline	2.48235	0.7659746

From WinStorm 3.04 Manual

The ratio of the side flow intercepted to total side flow, Rs, or side flow efficiency, is expressed by Equation 4.9

$$R_s = \left[1 + \frac{0.15z\,V^{1.8}}{L^{2.3}}\right]^{-1} \qquad (4.9)$$

where;

R_s = ratio of side flow intercepted to total flow
z = inverse of transverse slope, $(1/S_x)$
V = approach velocity of flow in gutter, ft/s
L = length of grate, ft

The efficiency of the grate is then calculated using Equation 4.10.

$$E_f = \left[R_f\,E_o + R_s\,(1 - E_o)\right] \qquad (4.10)$$

The interception of the grate inlet, Q_i, is given by the total flow of the grate, Q, multiplied by the efficiency of the grate, E_f. The bypass flow rate or overflow over the grate is then determined by the difference between the total flow, Q, and the interception of the grate inlet, Q_i.

4.4.1.5 Interception Capacity of Grate Inlets on Grade Example

Find: The interception capacity of a reticuline grate 2-foot square

Given: Flow in gutter – 1.47 cfs
                Velocity in gutter – 2.0 ft/sec
                Pavement cross slope, S_x – 0.03 ft/ft
                27 ft wide, back of curb to back of curb, residential street
                Longitudinal slope – 0.75%
                Roughness, n = 0.014
                Gutter width, 2 feet
                Water flow width, 7 feet

Solution: From Example 4.4.3 E_o = 0.5928

$$V_o = A\,L^N\,factor^{1-N}$$

$$V_o = (2.48235)\,(2^{0.7659746})\,(1)$$

V_o = 4.22 ft/sec

Since Velocity in the gutter = 2 ft/sec
which is less than V_o = 4.22 ft/sec R_f = 1.0.

$R_s = (1 + (0.15*z*V^{1.8})/L^{2.3})^{-1}$

$R_s = (1 + (0.15*1/0.03*2^{1.8})/2^{2.3})^{-1}$

$R_s = 0.2205$

$E_f = R_fE_o + R_s(1 - E_o)$

$E_f = 1(0.5928) + 0.2205(1-.5928)$

$E_f = 0.6826$

$Q_i = Q\ E_f$

$Q_i = 1.47$ cfs * 0.6826

$Q_i = 1.0$ cfs

4.4.2   INLETS IN A SAG

4.4.2.1 Grate Inlets in a Sag

Grate inlets constructed in a sag operate as a weir at low ponding depths and as an orifice at high ponding depths.   When the grate is operating as a weir the crest length is roughly equal to the outside perimeter (P) along which the flow enters. When the grate is operating as a weir the rate of discharge into the grate opening is calculated by Equation 4.11.

$$Q_i = 1.66P\ y^{1.5} \qquad\qquad (4.11)$$

where:

Q_i = rate of discharge into the grate opening, cfs
P = perimeter of grate opening, ft (disregarding the bars and neglecting the side against the curb)
y = depth of water at grate, ft

When the depth of flow becomes sufficient that the grate inlet is submerged then the grate begins to act like an orifice. The discharge for a grate inlet operating as an orifice is given by Equation 4.12.

$$Q_i = 0.67 \, A \, (2gy)^{1.5} \tag{4.12}$$

where:

Qi = rate of discharge into grate inlet, cfs
P = weir perimeter, ft
y = allowed ponded depth on grate, ft
A = clear opening grate area available for flow, ft^2
g = acceleration due to gravity, ft/s^2

The above equations do not account for clogging of the grate inlet. The designer should consider a clogging factor when determining the flow capacity of the grate inlet.

4.4.2.2 Interception Capacity of a Grate Inlet in a Sag Example

Find: The grate inlet size for a given design flow if:
  A. Grate inlet acts as a weir
  B. Grate inlets acts as an orifice

Given: Grate inlet with two sides against the curb (corner of parking lot)
  Flow from right – 3.2 cfs, flow from left – 2.5 cfs
  Longitudinal slope – 0.75%
  Roughness, n = 0.014
  Maximum water depth is 0.5 ft

Solution:

A. $Q = 1.66 \, P \, y^{1.5}$

1.47 cfs = 1.66 P $(.5 \text{ feet})^{1.5}$

P = 9.7 feet ; P = 12.95 feet w/ 0.75 clogging factor

B. $Q = 0.67 \, A \, (2gy)^{1.5}$

1.47 cfs = 0.67 A $(2*32.2*.5 \text{ feet})^{1.5}$

A = 1.5 ft^2 ; A = 2.0 ft^2 w/ 0.75 clogging factor

4.4.3   COMBINATION INLETS

Combination inlets consist of a curb inlet and a grate inlet placed side-by-side.   A combination inlet may also include part of a curb opening placed upstream of the grate. The curb opening intercepts debris that might clog the grate inlet.   Combination inlets may be used but they tend to not provide additional inlet capacity from curb inlets on grade or grate inlets.   In order to calculate the capacity of the combination inlet, it is recommended that the capacity of a standard curb on grade and a grate on grade calculated and the greatest capacity of the two inlet structures be used for the design.

4.5   STORM SEWER DESIGN

When designing the storm sewer system, the hydraulic gradient in the storm sewer shall be maintained at an elevation of half a foot below the elevation of the gutter elevation.   The minimum pipe size for the storm sewer shall be 18 inches. Bends or deflections are not allowed. Instead junction boxes with access points located in the top of the box shall be provided. Additional design criteria for Culverts in Section 3.2 applies as well.

4.5.1   SUBMITTAL INFORMATION

When submitting the design of a storm sewer system the storm sewer system shall be shown in plan and profile. The profile shall show the vertical alignment of the pipe as well as the hydraulic grade line of the flow. The hydraulic grade line shall not be higher than the pipe soffit.   The drainage report shall provide the design calculations for each inlet and all assumptions, variables, and coefficients needed for the design of the storm sewer system shall be summarized in tabular format.

4.6   WINSTORM

WinStorm is a computer program that can be used to calculate the capacity of inlets and storm drain systems.   The computer program can be downloaded from the Texas Department of Transportation web site and is free to the public. If WinStorm is used as the design aid, the complete report generated by WinStorm shall be submitted as part of the drainage report.   In addition, the layout of the analysis shall be included as well as a diskette containing the analysis.
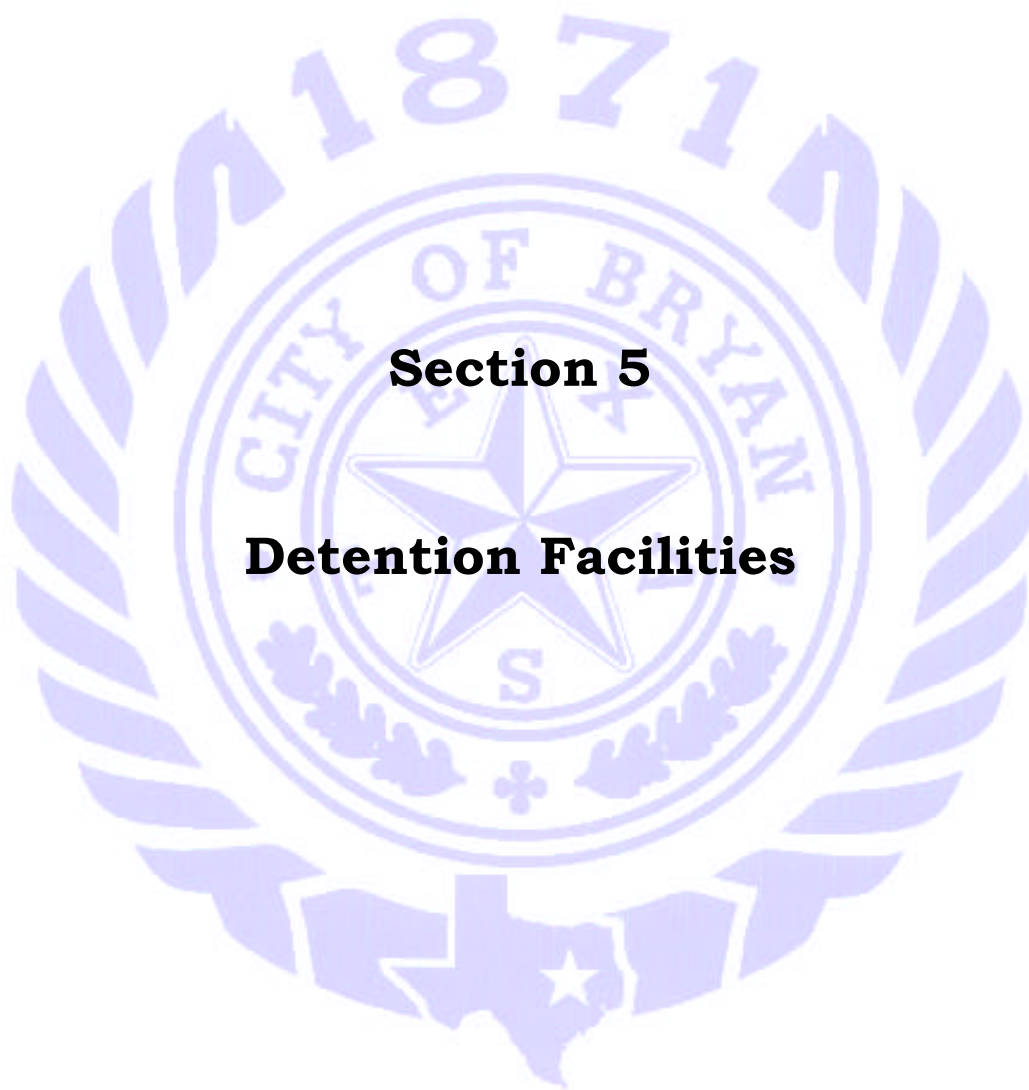
Section 5

Detention Facilities

5.1   INTRODUCTION

This section describes guidelines which are to be used in the design of detention facilities. The detention facility may include parking lots, surface detention facilities, or underground detention facilities. All detention facilities shall be designed to accommodate the corresponding design storms given in Section 5.2.3.1.

5.2   SURFACE DETENTION FACITLITIES

5.2.1   TYPES OF SURFACE STORAGE FACILITIES

Detention Facilities may be either on-line or off-line. Each facility may be designed to either retain or detain the storm water. A typical off-line detention facility captures storm water runoff directly from the development site and discharges into an adjacent drainage system.   If there is a tributary with a larger watershed that intersects the site, it's conveyance, even in the flooding event, does not provide any in-flow for the off-line facility.

An on-line detention facility is one where the tributary passes through the facility's outflow structure. Generally, an on-line detention facility is used to mitigate the storm water runoff from the site, which is being developed. However, the on-line facility may also provide a larger regional mitigation beyond the site development. Another type of on-line detention facility is one in which the runoff does not begin to flow into the detention facility until the discharge in the adjacent tributary reaches a critical level above which unacceptable downstream flooding occurs. This on-line facility is used to store only the runoff volume associated with the high flows associated with flooding events.

5.2.2   DESIGN CONSIDERATIONS

5.2.2.1 Location of Facility

If possible, detention facilities should be located near primary or secondary drainage systems and at the portion of the tract with lower elevations. The lower portion of the tract will enable the facility to collect the required runoff volume of water for the entire development. An off-line detention facility should not require off-site storm waters for adequate in-flows, but will be considered by the City Engineer in a case-by-case basis. Only minor to no encroachments into the floodplain fringe will be allowed for the facility.

Site grade breaks creating multiple watersheds may require multiple detention facilities. However, redirecting portions of the post-development flows may eliminate the need for multiple facilities.

5.2.2.2 Multi-Purpose Detention Facilities

In addition to parking lot detention facilities described in Section 5.4, detention facilities may serve other secondary functions such as parks or recreational areas. When a facility is proposed to serve as a park or recreational area, contact the City Engineer to discuss the possibility of the facility being contained within a Public Utility Easement for the City of Bryan to maintain. In addition the adequate access to the facility must be provided as described in Section 5.2.2.5.

5.2.2.3 Maintenance

When a detention facility is proposed to serve as a detention facility for a residential subdivision, the facility shall be contained within a Public Utility Easement and dedicated by plat to the City of Bryan. In this case, the detention facility will be required to meet geometric criteria, which allows for the City of Bryan to effectively maintain the facility. In addition the adequate access to the facility must be provided as described in Section 5.2.2.5.

If the facility is to be privately maintained, it shall be the responsibility of the property owner to ensure that the facility functions as it was designed and is kept operable in accordance with the associated drainage report and site plan. The facility will routinely be required to be cleared of debris, mowed, de-silted, and possibly re-vegetated. The facility is also subject to City inspection. All non-residential detention facilities shall be in dedicated private easements.

5.2.2.4 Erosion Control Measures

The construction of the detention facility is such that all of the existing vegetation is usually disturbed. This leaves the detention facility susceptible to increased erosion and siltation potential. In order to minimize this potential all public facilities shall have their side slopes block sodded as well as the top of the berm. If the geometry of the facility is such that there is no berm (i.e. a single side sloped facility) then the surrounding area of the facility within 10 feet of the top of the facility shall be block sodded. The detention facility should be a minimum of 80% vegetated.

In addition to the block sodding of the facility, a concrete lined pilot channel shall be provided in the bottom of the facility. The low flow pilot channel shall be constructed at

a minimum grade of 0.70%. Refer to the 4 foot wide concrete Standard Flume Section detail in the standard details.

Where a detention facility discharges into a nearby secondary or primary system channel, the discharge pipe shall enter the receiving channel at 45 degrees or less. In addition, the receiving channel shall be protected with baffled dissipaters and rip rap rock rubble as described in Section 3.2.5.2.5 to prevent the discharge from the facility from eroding the adjacent drainage channel.

5.2.2.5 *Access for Public Detention Facilities*

When a detention facility will be dedicated to the City of Bryan for future maintenance purposes, the facility shall be located such that adequate public access is provided. In general, if the detention facility is located away from a public street then a cleared 20-foot wide easement shall be provided for access to the facility. Since each development is unique and the location of the facility is different, the design engineer shall coordinate with the City of Bryan in the preliminary design of the development to ensure that adequate access is being provided.

5.2.2.6 *Geometry of Detention Facilities*

The geometry (see Figure 5.1 Typical Detention Facility and Figure 5.2 Typical Trapezoidal Wier) of a detention facility is important for maintenance purposes and to protect the integrity of the facility. All side slopes for the detention facility shall be a maximum of 3:1 for privately maintained facilities and a maximum of 4:1 for publicly maintained facilities. In addition, the top width of the berm shall be wide enough such that the integrity of the facility is maintained. Therefore the minimum berm widths shown in Table 5.1 shall apply to all detention facilities. The bottom slope of the detention facility shall be graded to a minimum slope of 0.70%.
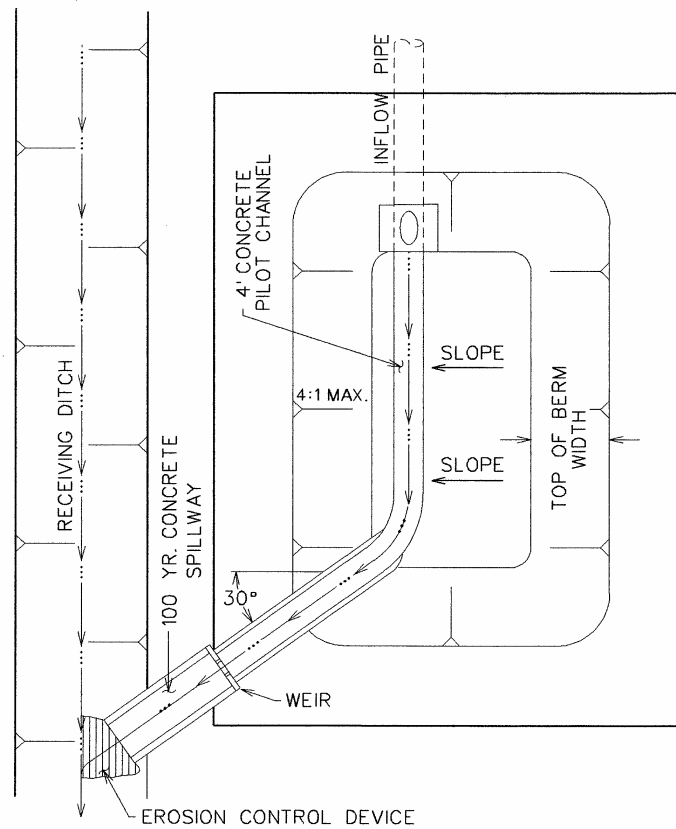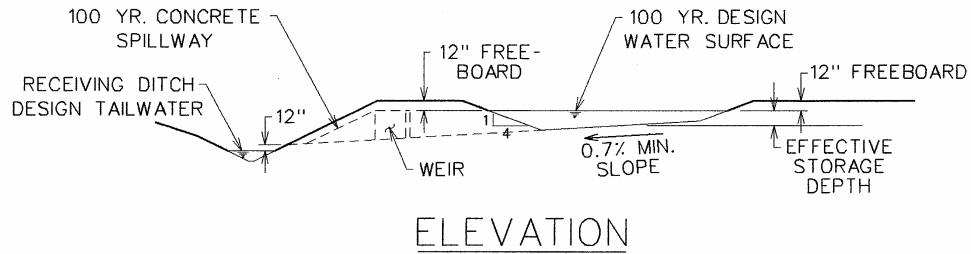
ELEVATION

PLAN

Figure 5.1 – Typical Detention Facility (City of Conroe, Texas)
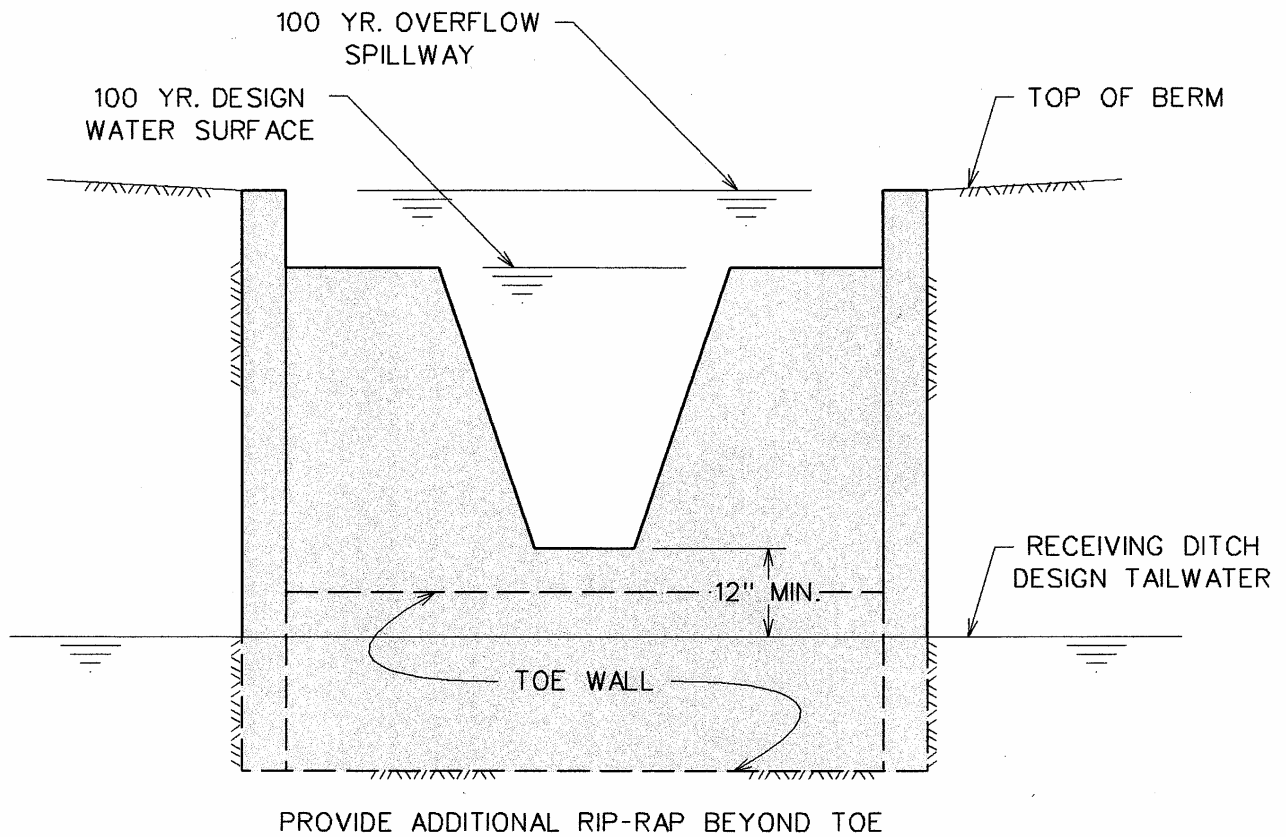
Figure 5.2 – Typical Trapezoidal Wier

Table 5.1 Minimum Top of Berm Widths

Depth of Facility	Required Top of Berm Width, feet
<= 2 feet	3
> 2 feet & < 6 feet	5
>= 6 feet	10

5.2.3   DESIGN PROCEDURES

5.2.3.1 Design Frequencies

All detention facilities shall be designed to attenuate developed conditions peak flow rates for each of the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year frequency storms, for a 24-hour duration storm to 90% of the runoff levels for the pre-developed conditions. The volume of the pond shall accommodate the 100-year storm (Table 1.1) with an additional one-foot freeboard in accordance with Section 5.2.3.4.4.

5.2.3.2 Design for Areas Less Than 50 Acres

The flow rate for these storm events shall be determined using the Rational Method described in Section 2.2. This method should only be utilized for off-line ponds. The volume of the facility shall be determined by using the triangular hydrograph method described in Section 2.2.5 and the example in Section 2.2.6.

The size of the outlet pipe shall be determined as described in Section 5.2.3.5.

5.2.3.3 Design for Areas of 50 to 640 Acres

For drainage areas greater than 50 acres but less than 640 acres as well as on-line facilities with contributing watersheds of less than 50 acres, an inflow hydrograph must be developed and routed through the detention facility. The hydrograph may be developed using the SCS Method or HEC-1.

The Modified Puls method of routing shall be used to route the flows through the detention facility. This method is described is described by Equation 5.1:

$$\frac{I_1 + I_2}{2} \Delta t + S_1 - \frac{O_1}{2} \Delta t = S_1 + \frac{O_2}{2} \Delta t \qquad (5.1)$$

where:

I = instantaneous inflow rate at the beginning of a routing period, cfs
O = instantaneous outflow rate at the beginning of a routing period, cfs
S = instantaneous storage volume at the beginning of a routing period, cfs
Δt = duration of routing period, seconds

The above equation may be used in a spreadsheet to route the flows through the detention facility or the use of other computer programs such, as HEC-1 are available. However, the submittal requirements set forth in section 5.2.3.6

5.2.3.4 Design for Areas Larger Than 640 Acres / On-line facility with >50-Acre Basins

For drainage areas larger than 640 acres and for any on-line facility with a contributing watershed of 50 acres or greater, the HEC-1 computer program will be used to analyze the proposed detention facility and to ensure that downstream water surface elevations do not exceed flooding levels. During the design phase, the engineer shall establish an existing conditions model in HEC-1 and submit this model to the City Engineer for approval. Once the existing conditions model has been established and agreed upon, the proposed development and detention facility shall be analyzed. All release rates described in Section 5.2.3.1.

5.2.3.5 *Outlet Structures*

The two primary types of structures that are acceptable for detention facility design are weirs and orifices. Several variations of each are provided below. Additionally, combinations of weirs and orifices may be utilized. Lastly, spillways and freeboard are also discussed. The flowline of the downstream end of the outlet structure shall be a minimum of 1 ft. above the normal water surface of the receiving drainage system. Outlet structures operating in submerged conditions for the 100-year storm event on the receiving drainage system shall be designed to accommodate both the submerged and the unsubmerged conditions.

Additional requirements may include a structural design on outlet structures with excessive loading due to traffic, structure height, soil conditions; as well as a private drainage easements will need to be acquired if a predevelopment sheet flow is proposed to be a point discharge near a property line.

5.2.3.5.1 Weirs

As discussed in Section 5.2.3.1 Design Frequencies, storms ranging from the 2-year to the 100-year will need to be attenuated. Restricting post-development flows for all the storms and maximizing the efficiency of the acreage (or volume) of the detention facility will often make the weir design the preferred choice.
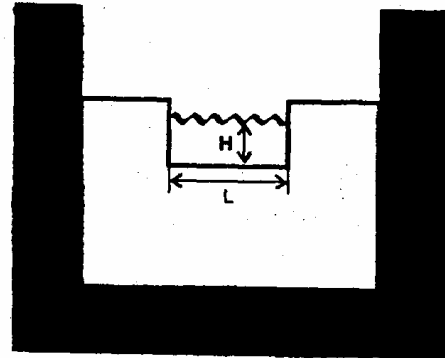
Additional advantages for the weir include the reduced trapping of debris, ease of debris removal, simplified spillway accommodation, and localized erosion protection. Typically, the V-Notch weir is difficult to construct in the field, however it may be utilized if design constraints exist.
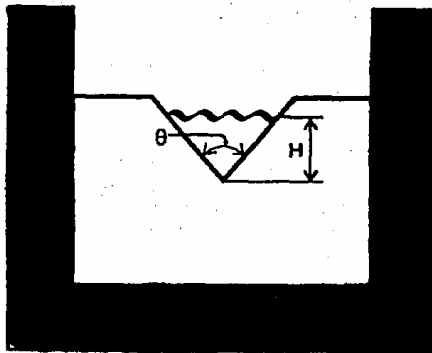
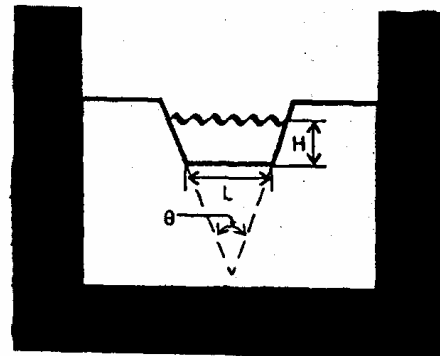Refer to Figure 5.2 – Typical Trapezoidal Wier for additional detail.
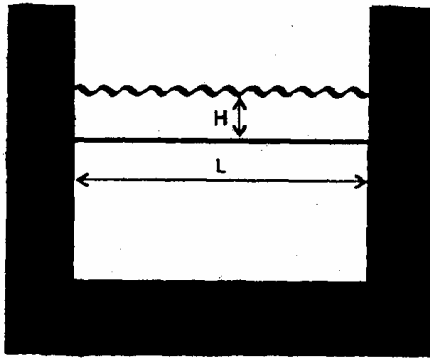
Rectangular Weir, Suppressed   Rectangular Weir, Contracted

Triangular Weir (V notch)   Trapezoidal Weir

Figure 5.2 Weirs – Four Types (Source – EaglePoint, Inc.)

Rectangular Weir, Suppressed:

$$Q = C_W L \, H^{1.5} \qquad (5.2)$$

Rectangular Weir, Contracted:

$$Q = C_W (L - 0.2H) \, H^{b} \qquad (5.3)$$

V-Notch Weir:

$$Q = C_w \tan(q/2) \, H^b \tag{5.4}$$

Trapezoidal Weir:

$$Q = C_w [L + 0.8H \tan(q/2)] \, H^b \tag{5.5}$$

where:

Q = outlet flow, cfs
C_w = weir coefficient (ranges from 2.5 to 3.1, typically 2.6 for rectangular and 2.9 for v-notch and trapezoidal)
L = crest length, ft
H = head on wier to crest, ft (refer to *h* in Orifice section for submerged)
Q = internal angle (radians)
b = constant (typically 1.5)

5.2.3.5.2 Orifices

Minimum outlet pipe size shall be 18 inches for all detention facilities. Facilities to be publicly maintained shall have discharge pipes constructed of reinforced concrete pipe. If an orifice design requires an open area smaller than an 18 inch, a smaller structure should be constructed on the upstream end of an 18 inch or larger RCP to better facilitate the removal of debris. Multiple orifices and/or culverts may be utilized to provide a design that attenuates all required design storms. All facilities utilizing culverts shall have concrete headwalls on each end.
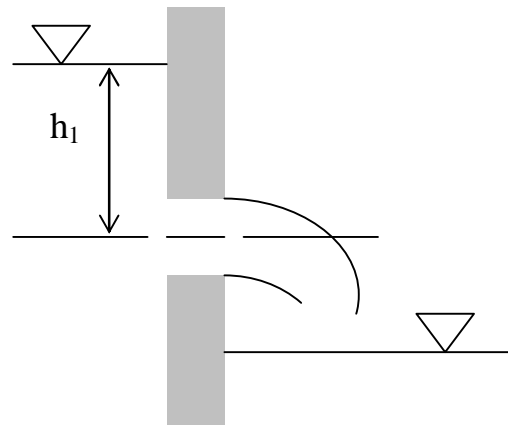
$$Q = C \, A \sqrt{2gh_1} \tag{5.6}$$

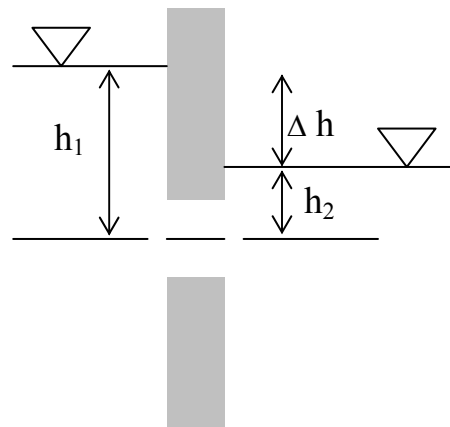Figure 5.4 - Vertical Orifice, Unsubmerged

$$Q = C\,A\sqrt{2g(h_1 - h_2)}$$

$$Q = C\,A\sqrt{2g\Delta h} \qquad (5.7)$$
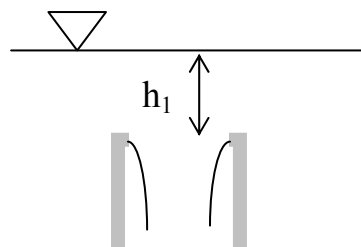
Figure 5.5 - Vertical Orifice, Submerged

$$Q = C\,A\sqrt{2gh_1} \qquad (5.8)$$

Figure 5.6 - Horizontal Orifice (Stand Pipe)

where:

Q = outlet flow, cfs
C = coefficient (0.6 for extruded and 0.8 for flush mount)
A = open area, sf.
g = acceleration due to gravity, ft/s^2
h_1 = head on upstream side of orifice to orifice centroid, ft
h_2 = head on downstream side of submerged orifice to orifice centroid, ft
Δh = head on submerged orifice, ft

5.2.3.5.3 Spillways

All detention facilities shall have a concrete overflow spillway to protect the integrity of the detention facility. The spillway shall be located at the elevation of the 100-year storm event to provide a depth of conveyance of 1-foot through the spillway.

5.2.3.5.4 Freeboard

The maximum 100-year water surface elevation in all detention facilities shall be a minimum of 1 foot below the top of bank elevation of the facility.

5.2.3.6 Additional Submittal Requirements

Refer to Section 1.4.1 for standard Drainage Report Requirements as well as Section 1.6 for As-Builts and certifications.

All routing calculations shall be provided within the drainage report submitted to the City of Bryan. The report shall contain the dimension of the detention facility along with contours at 1-foot intervals. The side slope dimensions as well as the length, and width of top of the facility shall be shown. In addition, included in the report shall be a table showing the contour interval and the storage volume of the detention facility at that depth. For the detention facility outfall, all details for the outlet structure shall be shown with dimensions. Reinforcing requirements for concrete structures shall also be shown. Energy dissipaters needed in the outfall structure shall be shown as well as a detail showing how the dissipaters connect to the concrete apron. All inflow and outflow hydrographs for each storm event shall be shown. A summary table shall be provided showing the inflow flow rate and the maximum outflow flow rate for each storm event.

Any computer program printouts will not stand alone as a drainage report. The engineer may supply this information in an Appendix however all information supplied to meet the requirements of the drainage report shall be contained within the body of the report. Exceptions to this include drawings of the site and drawing design details as well as hydrographs. However, computer printouts, which incorporate a graph by the use of symbols or characters such as "x", "o", and "y" are not permitted. The information generated by this program shall be incorporated into a chart in which a continuous line is plotted.

5.3   SUBSURFACE DETENTION FACILITIES

Where a development is required to provide storage for stormwater runoff and is limited on surface area, subsurface detention facilities are acceptable. The subsurface facilities shall be designed to provide a storage volume and limit discharge rates to those volumes and rates described within Section 5.2. In addition, the facility shall be privately maintained and shall not be dedicated to the City of Bryan for maintenance purposes.

5.4   PARKING LOT DETENTION

Although not encouraged due to general pedestrian inconvenience, an on-site parking lot may be used as a private detention facility. The possible depth of the water in the parking lot shall not at any time exceed nine (9) inches. A spillway accommodating a minimum of the 100-year storm must be provided at an elevation not exceeding the described nine inches in depth. The outlet structure is commonly a horizontal grate sized with the Horizontal Orifice Equation 5.8. The release rate from the detention facility shall be consistent with the guidelines set forth in Section 5.2.3 of this Manual.

5.5   PUMP DETENTION FACILITIES

Detention facilities, which require a pump to discharge the stored stormwater runoff shall not be considered as a design alternative or allowed in the City of Bryan.
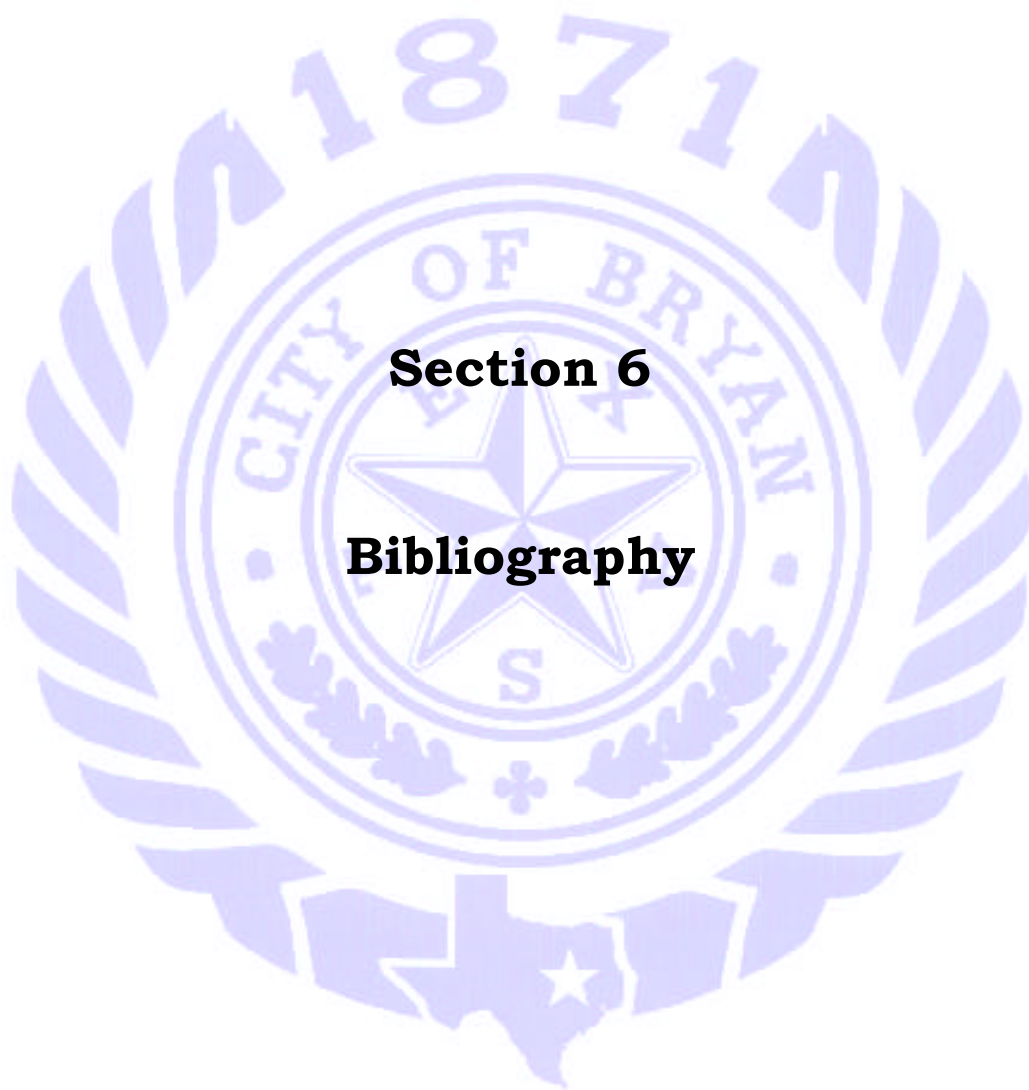
Section 6

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